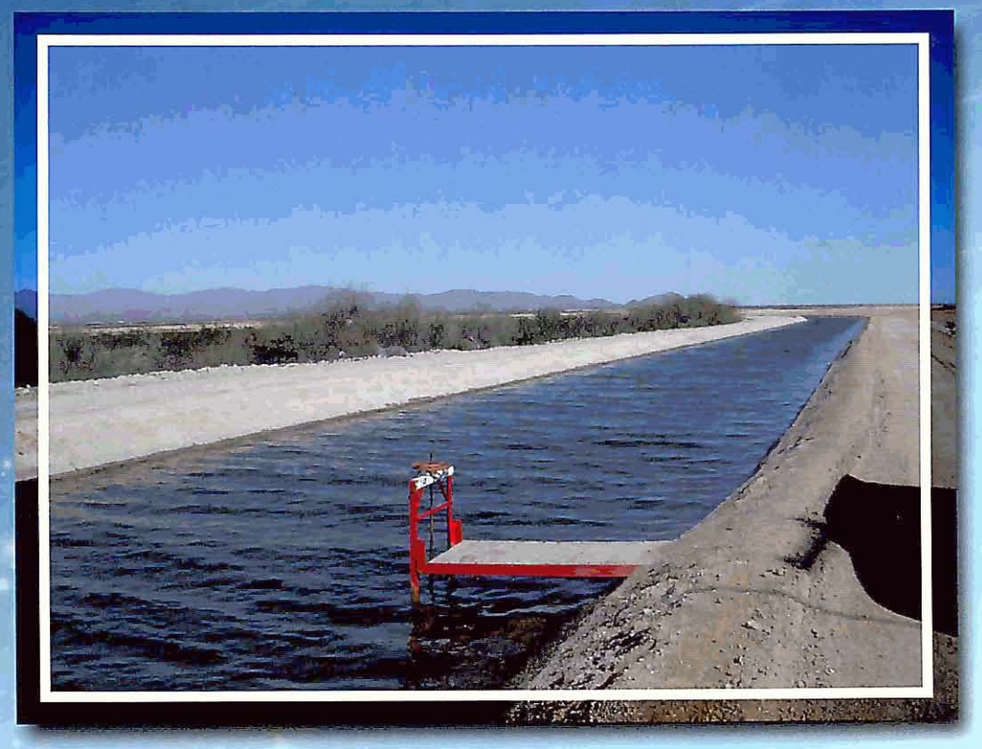
JULY 2005

BUCKEYE/SUN VALLEY AREA DRAINAGE MASTER STUDY
TECHNICAL DATA NOTEBOOK

CONTRACT FCD 2002C027

BUCKEYE FLOOD RETARDING STRUCTURES
PROBABLE MAXIMUM FLOOD (PMF)
HYDROLOGY & HYDRAULICS
REVISED



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Buckeye/Sun Valley Area Drainage Master Study

Contract FCD 2002C027

Technical Data Notebook

BUCKEYE FLOOD RETARDING STRUCTURES PROBABLE MAXIMUM FLOOD (PMF) HYDROLOGY & HYDRAULICS

(Revised)

Prepared for

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(Revised)

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1 Introduction

1.1 Purpose of the Study

The Flood Control District of Maricopa County (District) contracted with PBS&J to perform the Buckeye/Sun Valley Area Drainage Master Study (ADMS) to identify drainage, erosion, and sediment problems; evaluate existing floodplain delineations and delineate addition floodplains; identify a range of implementable solutions; and develop preliminary development guidelines for the area. The study area encompasses 280 square miles and is divided into four distinct hydrologic regions. Area 3, north of I-10, is the tributary watershed for the Buckeye Flood Retarding Structures (FRS) #1-3 and is approximately 90 square miles in size. The Buckeye Structures are located north of and parallel to Interstate 10 (I-10).

The purpose of the hydrologic analysis is to develop hydrology for Area 3. The results will be used subsequently to obtain frequency-stage relationships for the FRSs. Another purpose of this study is to provide updated hydrologic models for both the existing and future conditions; specifically; this study will update the current hydrologic study (Alpha Engineering, 1996). The updated hydrology will be used to determine existing and future drainage problems and to develop alternatives to mitigate those identified problems.

The purpose of the hydraulic analysis is to quantify the extent of overtopping by various storm events up to the probable maximum flood (PMF) and develop dynamic stage-frequency relationships for the Buckeye Structures to facilitate dam management and risk assessment. Because the Buckeye Structures are very long, the use of level pool routing methods to represent system response during major storms has been questioned. Dynamic routing methods are believed to provide a better assessment of flooding conditions for long dams; however, the modeling is more complex and labor-intensive. As a result, the District has requested that PBS&J evaluate the operation of the Buckeye Structures using an alternative dynamic routing approach.

This is accomplished by conducting an unsteady flow analysis of FRS hydraulics using the U.S. Army Corps of Engineers HEC-RAS (Version 3.1.1) computer program. Associated with the FRS hydraulics analysis, downstream inundation mapping for emergency spillway discharges of the Buckeye Structures from the spillway #2 and #3 to the Roosevelt Irrigation District (RID) Canal are developed. The emergency spillway downstream inundation analyses are included in a separate report. Another objective is to determine whether the HEC-RAS model can provide a cost-effective alternative to two-dimensional modeling for this type of study.

1.2 Project Authorization

A notice to proceed (NTP) for this study was issued in June 2003 by the District. The FCD Contract Number for this project is 2002C027.

1.3 Location of Study

Area 3 covers all or part of sections in Township 1N to 3N, Range 3W to 5W, located in the Town of Buckeye, Arizona. A project location map is presented in Figure 1-1. Area 3 is bounded on the south by I-10 and is located between the White Tank Mountains and the Hassayampa River.

1.4 Methodology Used for Hydrology and Hydraulics

The previous hydrology model of the area (Alpha, 1996) was used as a basis for and comparison to the results produced in this study. HEC-1 embedded in WMS version 7.0 (September, 2003) was used to model the hydrologic response of the watershed for eight storm events ranging from the 10-year, 6-hour storm to the PMP event. The HEC-1 parameters were determined using the methods described in the Drainage Design Manual for Maricopa County, Arizona, Volume I, Hydrology, hereinafter referred to as the Hydrology Manual (Sabol et al., 1995). The Soil Conservation Service (SCS, now Natural Resource Conservation Service - NRCS) principal spillway hydrograph (PSH) method (NRCS, 1975) was used to model the hydrologic response of Area 3 to the 100-year, 10-day storm event.

The unsteady flow module of HEC-RAS version 3.1.1 (May, 2003) was used to model the hydraulic response of the Buckeye Structures for all of the various storm events. The HEC-RAS model parameters were determined using the methods described in the Drainage Design Manual for Maricopa County, Arizona, Volume II, Hydraulics, hereinafter referred to as the Hydraulics Manual (Sabol et al., 1995).

1.5 Acknowledgements

Ms. Valerie Swick, CFM (Project Manager, Flood Control District of Maricopa County) provided guidance during the course of study. Dr. Bing Zhao, P.E. (Engineering Application Development Branch Manager, Engineering Division, Flood Control District of Maricopa County) and Mr. Joe Rumann, P.E. (Sr. Hydrologist, Engineering Division, Flood Control District of Maricopa County) provided advice on technical issues. Mr. Brett Howey, P.E. (Dam Safety Engineer, Planning & Project Management Division, Flood Control District of Maricopa County) provided coordination with the Arizona Department of Water Resources (ADWR). Ms. Laurie Miller, P.E. (LTM Engineering, Inc.) assisted with the report. Dr. Mike Johnson, P.E. (Dam Safety Engineer, Arizona Department of Water Resource) contributed discussions about dam overtopping policy in Arizona. Saul Nuccitelli, P.E., Michael Depue, P.E., and Bob Laura, P.E., PBS&J reviewed the model results and suggested debugging techniques.

1.6 Summary of Study Results

Eight storm events (both existing and future conditions) were simulated in the HEC-RAS unsteady models. The results were compared to level pool routing results. A summary of the hydraulic results is presented below:

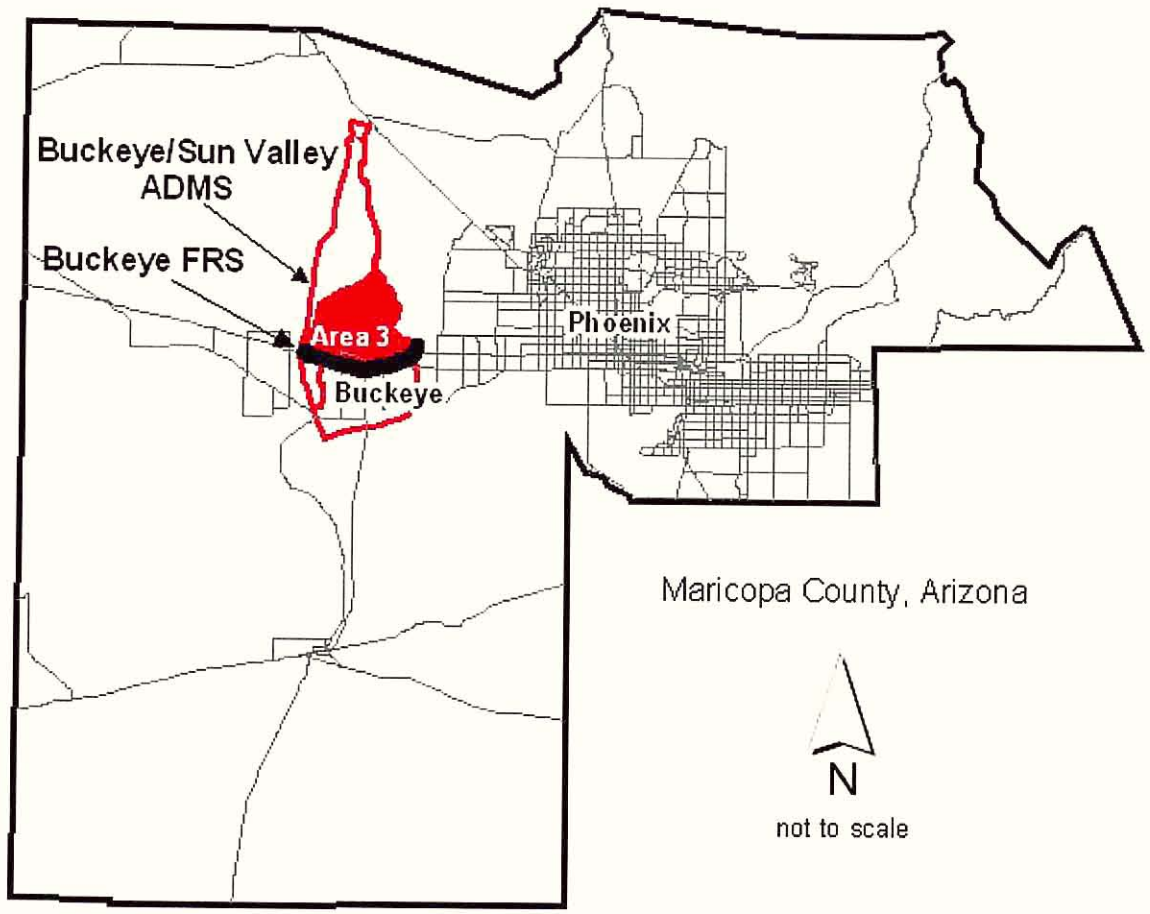
- The original FRS design can safely contain and convey runoff for storms up to the 500-year event without overtopping the emergency spillways.
- Overtopping of the FRS crest during the Probable Maximum Flood (PMF) occurs only on portions of FRS#1 and FRS#2.
- Emergency spillways for all FRSs are overtopped during the PMF, however, the discharge capacity of the spillways is not fully utilized.
- The unsteady model analyses can identify problems with existing associated structures (roadways and culverts, principal spillway, etc.) that are ignored in less sophisticated models.

Warning messages during the analyses indicated floodways were subject to discharge during a PMF event. Additional analyses with floodway overtopping were conducted. The results highlight the following bullets for future dam rehabilitation.

- The original analyses provide the minimum depths to be leveled for the floodways to avoid overtopped at a PMF event.
- The additional analyses indicate the maximum overtopping discharges for the floodways, which could be a weak link in the Buckeye FRS system.
- Transverse roadways affect the FRS hydraulic characteristics significantly. More discharge could be expected at the emergency spillway if the roadway did not exist.
- The floodway discharge does not affect the emergency spillway hydraulic profile.

Figure 1-1

Buckeye Area 3 ADMS Vicinity Map



2 Study Abstract

2.1 General Information

2.1.1 Community Name

The Town of Buckeye, Arizona and unincorporated areas of Maricopa County.

2.1.2 Community Number

040037

2.1.3 County

Maricopa

2.1.4 State

Arizona

2.1.5 Date Study Accepted

July 2004

2.1.6 Study Contractor

PBS&J, 7310 N. 16th Street, Suite 310, Phoenix, Arizona, 85020.
Technical contact: Frank Turek

2.1.7 State Technical Reviewer

Arizona Department of Water Resources (ADWR)

2.1.8 Local Technical Reviewer

Flood Control District of Maricopa County (The District)

2.1.9 River or Stream Name

Hassayampa River and White Tank Wash

2.1.10 Reach Description

The Buckeye FRSs were designed and constructed by the NRCS in the 1970s and have a combined contributing drainage area of 90 mi^2. The three FRSs were designed to function as a single system with the detained flows cascading west to the outfall at the Hassayampa River. The structures were designed to provide protection from storms up to the one percent exceedance level. Ungated low-level outlets serve as principal spillways and runoff to the west. Free surface emergency spillways were provided for each FRS to control discharge for storms in excess of a storm event equivalent to "74% of the 100-year storm plus 26% of the probable maximum flood (PMF)". The dam crest, or "emergency spillway freeboard", was designed to contain the PMF without overtopping (NRCS, 1985; Arizona Water Commission, 1979). The principal outlet discharges are routed to each downstream FRS through Buckeye Floodways #2 and #3. Emergency spillway discharges do not cascade to other FRSs but flow generally to the south through the Town of Buckeye toward the Gila River.

2.1.11 Study Type

Detailed hydrology; and FRS frequency-stage study.

2.2 Mapping Information

2.2.1 USGS Quad Sheets

N/A

2.2.2 Mapping for Hydrology Study

Ten-foot contours derived from a digital elevation model (DEM) built from mass points on a 50-foot grid and breaklines and one-foot resolution MrSID format orthophotography provided by the District in 2003 were used as the base mapping for the Area 3 hydrology study.

2.2.3 Mapping for Hydraulic Study

Two-foot contours derived from the 50-foot mass points and breaklines digital elevation model (DEM) and one-foot resolution MrSID format orthophotography provided by the District in 2003 were used as the base mapping for the Area 3 hydraulic study.

2.3 Hydrology

2.3.1 Model or Method Used

The US Army Corps of Engineers HEC-1 program was used for this study. Parameters included Green & Ampt rainfall/runoff losses with S-graph unit hydrographs and normal depth routing using 8-point cross sections.

2.3.2 Storm Frequency and Duration

For both existing and future conditions, the following storm frequencies and durations were analyzed:

- 10-year 6-hour
- 10-year 24-hour
- 100-year 6-hour
- 100-year 24-hour
- 100-year 10-day
- 200-year 24-hour
- 500-year 24-hour
- PMP 6-hour
- PMP 72-hour

However, in this report to ADWR, only the PMP results are included.

2.3.3 Hydrograph Type

S-graphs (Mountain and Desert/Rangeland) developed for Maricopa County.

2.3.4 Frequencies Determined

10-year, 100-year, 200-year, 500-year, and PMP

2.3.5 List of Gages used in Frequency Analysis or Calibration

N/A

2.3.6 Rainfall Amounts and Reference

The rainfall amounts were derived from the NOAA Atlas 2 isopluvial maps as provided in the Hydrology Manual for events up to the 100-year return frequency storm. For those events greater than the 100-year return frequency, the rainfall amounts were derived from a linear extension of the Intensity-Duration-Frequency (I-D-F) relationship in a normal-logarithmic scale (Prefre, ver. 2.1 in DDMSW). PMP amounts were derived based upon the procedures described in the National Weather Service (NWS) Technical Report HMR-49: *Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages* (NWS, 1977).

2.3.7 Unique Conditions and Problems

Special problems encountered in the course of study are discussed in subsequent sections, including:

- Unit hydrograph K_n-values for desert/rangeland
- Future condition assumptions used for hydrology modeling

2.3.8 Coordination of Q's

Peak flows for Area 3 submitted in this report have been coordinated with Julie Cox (hydrologist, Engineering Division in the Flood Control District of Maricopa County) throughout the entire project. The hydrologic models were submitted for review to Ms. Cox in October and December 2003 and revised with Ms. Cox's comments. The final 100-year storm event peak flows from the PBS&J study were comparable (within 15%) to Alpha's previous study.

2.4 Hydraulics

2.4.1 Model or Method Used

The unsteady flow module established in HEC-RAS (version 3.1.1) developed by the Corps of Engineers (May 2003).

2.4.2 Regime

The flow regime in the unsteady hydraulics model is assumed subcritical for all reaches. Starting boundary conditions for the unsteady models including time steps, initial flows, WSEL profile, and sediment volumes are set up using the default settings in the program with engineering judgments.

2.4.3 Frequencies for which Profiles were computed

For both existing and future conditions, the following storm frequencies and durations were used:

- 10-year 6-hour
- 10-year 24-hour
- 100-year 6-hour
- 100-year 24-hour
- 200-year 24-hour
- 500-year 24-hour
- PMP 6-hour
- PMP 72-hour

However, in this report to ADWR, only the PMP results are included.

2.4.4 Method of Floodway Calculation

N/A

2.4.5 Unique Conditions and Problems

Special problems encountered in the course of study are discussed in subsequent sections, including:

- Comparison of the level pool routing with unsteady hydraulic model results
- Floodway overtopping during the PMF events
- Modeling of roadways transverse to the FRS structure
- Modeling of the emergency spillways and dam crest for overtopping
- Development of suitable rating curves
- Low flow conditions

2.5 Additional Study Information

N/A

3 Mapping and Survey Information

3.1 Field Survey Information

3.1.1 Hydrology

The first field investigation was conducted on June 20, 2003 to scope out the critical points of the watershed and to identify problematic areas. The first investigation was conducted on November 11 and 12, 2003 after the sub-basin boundaries were delineated and the physical parameters for each basin were set. Major culverts along the Sun Valley Parkway were surveyed and measured. The results show that as much as 50% of the culvert conveyance area is blocked by sediments accumulated upstream, downstream, and within the culverts. The sediments are mostly sand and gravel with some cobbles. Many of the culverts are also obstructed by dense weeds, brush, and trees. Many of the trees are mature and much taller than the culvert (Appendix C, Picture #1 - #47). All of these factors will cumulatively decrease the culvert conveyance capacity and impair its hydraulic function. Boundaries between sub-basins were surveyed and no visible evidence of diversion was found (Appendix C, Picture #48- #60).

The second investigation was conducted on November 18, 2003 for channel routings. Only routing channels which were significantly different from that of the previous study (Alpha, 1996) were investigated, which included a typical 8-point cross section measurement and estimation of Manning's "*n*" for left overbank (LOB), Main channel, and right overbank (ROB) (Appendix C, Picture #61-#75). The results of the routing channel measurements are summarized in Tables D.3-1 and D.3-2 in Appendix D. These data were used for basin lag time calculations.

3.1.2 Hydraulics

Field investigations were conducted by PBS&J in February 2004. Additional information was collected from the District in 2002 – 2003 and was incorporated into this study.

The PBS&J field investigation mainly focused on the hydraulic elements identification, roughness estimation, and infrastructure updates and modifications as compared to asbuilt plans. For example, the culverts beneath Sun Valley Parkway were originally designed as two 60'' diameter corrugated metal pipes (CMP) (NRCS, 1974), but were replaced with four 10'X10' box culverts. The fields visit photographs and descriptions are documented in Appendix C.

The District field investigations were conducted in 2002 and 2003. These investigations are a part of the District's Dam Safety Monitoring Program. These photographs and survey results were mainly used to update the dam crest and spillway elevations. The original design assigned one constant elevation for each FRS dam crest (NRCS, 1974). Photographs provided by the District (Mr. Larry Lambert, Project Manager – Dam Safety, Planning and Project Management Division) with descriptions are documented in Appendix C.

3.2 Mapping

3.2.1 Hydrology

New mapping for Buckeye Areas 1 and 2 and the southern boundary of Area 3 (the three FRSs) was prepared for the District by Kenney Areal Mapping, Inc. in 2003. The new mapping provided 2-foot contour data, which are much more accurate than the USGS 7.5 digital elevation model (DEM).

Areal photography was flown in December 2002 as part of the District's ongoing effort to maintain current areal photography for the entire county. The areal photography for the study area is 2003 1-foot resolution orthophotography in Mr. SID format.

Mapping for the entire study area was prepared for the District by LandData Inc. in 2000. This mapping was prepared with 10-foot grid data derived from 50-foot mass points and break lines, and provides 10-foot contour elevations.

The District has some older strip mapping of the Buckeye FRSs survey prepared by McLain Harbers Co., Inc. in 1995. This strip mapping was 2-foot contour mapping.

The projected coordinates of State Plane NAD 1983, Arizona Central are used as the horizontal datum and the projected coordinates of NAVD 1988 are used as the vertical datum for all of the mapping.

3.2.2 Hydraulics

The District has some older strip mapping of the Buckeye FRSs survey prepared by McLain Harbers Co., Inc. in 1995. This strip mapping was 2-foot contour mapping.

Mapping for the entire study area was prepared for the District by LandData Inc. in 2000. This mapping was prepared with 10-foot grid data derived from 50-foot mass points and break lines and provided 10-foot contour elevations.

New topographic mapping for the southern boundary of Area 3 was prepared for the District by Kenney Aerial Mapping, Inc. in 2003. The new mapping provided 2-foot contour data. Aerial photography was flown in December 2002 as part of the District's ongoing effort to maintain current aerial photography for the entire county. The aerial photography for the study area is 1-foot resolution orthophotography in MrSID format.

The projected coordinates of State Plane NAD 1983, Arizona Central, were used as the horizontal datum and the projected coordinates of NAVD 1988 were used as the vertical datum for all of the mapping.

Vertical data including ground profile and water surface elevation (WSEL) presented in the report use NAVD88 as reference datum unless stated differently.

4 Hydrology

4.1 Method Description

The hydrologic responses of Area 3 to the various storm events were simulated using the modified U.S. Army Corps of Engineers HEC-1 (June 1998) computer program embedded in WMS 7.0 (September, 2003). The HEC-1 parameters were determined using the methods described in the Hydrology Manual (Sabol et al., 1995). The computational interval used to determine the unit hydrograph for each sub-basin is 5 minutes.

4.2 Parameter Estimation

This section includes a complete description of the methodology and calculations used to develop the hydrology.

4.2.1 Drainage Area Boundaries

Area 3 of the Buckeye/Sun Valley ADMS is defined as the area that drains to the Buckeye FRSs. The area begins at the west side of the White Tank Mountains and is approximately 90 square miles. The water flows in a south to southwest direction toward the Buckeye FRSs.

The three structures were designed for a 100-year event and drain west toward the Hassayampa River. Currently, this watershed is natural desert with several identified alluvial fans. A number of master planned communities are planned within this area. Sub-basins A through C do not contribute to the Buckeye FRSs, but are included in Area 3 hydrology for consistency in the updating of Alpha's study (1996). Figure 4-1a and 4-1b (Pocket) show the drainage area boundaries, major sub-basin boundaries, and concentration points, for both existing and future conditions.

4.2.2 Watershed Work Maps

The watershed work maps include:

- General watershed work map with the aerial photography and contour lines as background (Figure 4-1, in Pocket);
- HEC-1 schematic map (Figures 4-1a & b for existing and future conditions, in Pocket);
- Drainage basin map showing lag flow paths, and routing reaches (Figure 4-2, in Pocket);
- Soil map (Figure 4-3, in Pocket);
- Land use maps (Figures 4-4a & b for existing and future conditions, in Pocket);
- Flow maps including the drainage basin map with lag time paths (Figures 4-5a & b, for existing and future conditions, in Pocket).

4.2.3 Gage Data

There are no existing precipitation gages or stage gages within the study watershed. There are precipitation and reservoir pool gages at the downstream end of the watershed at each of the three FRSs. There is also a weather station located in the Hassayampa River at the I-10 bridge.

4.2.4 Statistical Parameters

The statistical parameters such as rainfall depths and rainfall distributions for storms up to the 100-year return frequency events are based on the data provided in the Hydrology Manual (Sabol et al., 1995). PMP amounts are derived based upon the data provided in NWS Technical Report HMR-49: Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages (NWS, 1977).

4.2.5 Precipitation

4.2.5.1 Rainfall Depths

The NOAA Atlas 2 data set is used for point rainfall amounts in this study. The NOAA Atlas 14 data set is available for use; however, it has not been accepted to date by the District. For the same storm event, rainfall depths in NOAA 14 are approximately 15 to 25% less than that in NOAA 2.

The point rainfall depths for less than and equal to 100-year, 24-hour storm events are obtained from the isopluvial maps in the Hydrology Manual (Sabol et al., 1995). For those storms above the 100-year return frequency event, the rainfall depths are derived from a linear extension of Intensity-Duration-Frequency (I-D-F) relationship in a normal-logarithmic scale (Prefre, ver. 2.1 in DDMSW) (Appendix D.1). PMP amounts are derived based upon the procedure in NWS Technical Report HMR-49: *Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages* (NWS, 1977).

The rainfall depths presented in Table 4-1 are area-weighted based on the watershed contributing to each FRS respectively.

Table 4-1
Summary of Rainfall Depths for Area 3

	FRS 1	FRS 2	FRS 3
Storm Event	Depth [in]	Depth [in]	Depth [in]
10-yr, 6-hr	2.08	2.07	2.07
10-yr, 24-hr	2.62	2.56	2.58
100-yr, 6-hr	3.23	3.28	3.26
100-yr, 24-hr	4.16	4.13	4.14
200-yr, 24-hr	4.63	4.61	4.62
500-yr, 24-hr	5.23	5.22	5.23
PMP, 6-hr	9.8	13.6	12.8
PMP, 72-hr	15.8	17.5	17.5

Rainfall amounts for PMP events are average values based on the watershed contributing to each FRS respectively. Therefore, no areal reduction factor is used for PMP HEC-1 models.

4.2.5.2 Rainfall Distributions

The HEC-1 computer models require the PMP distribution to be in 5-min intervals. HMR-49 lists only the 6-hour PMP distribution for 1-hour intervals. To transform the 1-hr into a 5-min interval distribution, the method developed in "*Phase I Report: Project Calculations - Hydrologic Analysis - Buckeye Floodwater Retarding Structures #1, #2, and #3*" by Dames & Moore (1990) was followed. Basically the method used EM110-2-1411 distribution pattern. PMP rainfall distributions for each FRS are presented in Figures D.1-2 and D.1-3 for 6-hour and 72-hour events, respectively (Appendix D.1). The values of rainfall distributions can be found in the final HEC-1 models (Appendix D.6).

4.2.6 Physical Parameters

This section describes the methods used for estimation of the physical hydrologic parameters such as rainfall losses, the unit hydrograph, and lag time for the desert/rangeland type terrain drainage area.

4.2.6.1 Rainfall Losses

The Green-Ampt procedure is used to compute rainfall losses in accordance with the Hydrology Manual (Sabol et al., 1995). The digital soil file and land use files (both in GIS shape file format) for the study area were provided by the District. The digital soil file corresponds to SCS soil maps (Camp, 1986). The GIS land use shape file is based on published digital GIS land use maps (MAG, 2000), the most current land use information. Figures 4-3 and 4-4a & b describe the soils and land use, respectively. An apparent discrepancy for the same soil texture located in the vicinity of Township 1N in Area 3 (Figure 4-3) is due to two separate soil survey results. An average value from the two soil survey map units is used for this study.

The GIS land use shape files and soil files are imported into WMS 7.0. Two additional attribute tables (one for soil and another for land use, Tables D.2-1 and D.2-2) are imported into WMS 7.0 for the Green-Ampt rainfall loss parameter estimation. The attribute tables for the land use types can be found in Appendix D.2.

The Green-Ampt rainfall loss parameters used in the HEC-1 input files (Appendix D.6) can be found in Tables D.2-3 and D.2-4 for existing and future conditions, respectively. For future conditions, the Green-Ampt parameters are estimated based upon variations of land use types according to conceptual community plans within the study area. This study assumes that the Green & Ampt loss parameters do not change with storm return frequencies.

Five major land uses are identified for the watershed, which are vacant, estate residential (1/5 dwelling per acre to 1 dwelling per acre), rural residential (<1/5 dwelling per acre), large lot residential/single family (1 dwelling per acre to 2 dwellings per acre), and water (MAG, 2000). The surface retention losses, percent impervious, and vegetation cover for each land use type are

estimated based on the Hydrology Manual (Sabol et al., 1995) and MAG (2000). The values of these parameters are shown in Table 4-2, which is a subset of Table D.2-1a in Appendix D.2.

Table 4-2

Land Use Surface Retention Losses, Percent Impervious, and Vegetation Cover

Land Uses	Surface Retention Losses: IA (inch)	Percent Impervious RTIMP (%)	Vegetation Cover (%)
Vacant	0.4	0.0	25.0
Estate Residential	0.3	5.0	30.0
Rural Residential	0.3	5.0	30.0
Large Lot Residential/Single Family	0.3	15.0	50.0
Water	0.0	100.0	100.0

4.2.6.2 Unit Hydrograph

The Maricopa County S-Graph method is used to compute unit hydrographs in accordance with the Hydrology Manual (Sabol et al., 1995), because for routing rainfall excess in Maricopa County, the recommended procedures are either the Clark Unit Hydrograph or the application of selected S-graphs; the Clark Unit Hydrograph procedure is more appropriate for watersheds less than about 5 square miles in size with an upper limit of application of 10 square miles and is preferred for urban watersheds, while the Area 3 watershed is approximately 90 square miles and is characterized as desert/agriculture land type; and S-graph method was used in Alpha's study (1996).

The Phoenix Mountain and Desert/Rangeland S-Graphs are used to compute the unit hydrographs. Alpha (1996) used Phoenix Valley S-graph for desert area because the Desert S-Graph was still under development. The unit hydrographs are computed by using WMS 7.0. The unit hydrograph can be found in the UI record in the HEC-1 input files (see Appendix D.6).

4.2.6.3 Lag Time

Lag time must be estimated before S-Graph unit hydrographs can be computed. Four parameters must be computed to estimate the lag time for each sub-basin. These four parameters are the longest watercourse length measured from the concentration point to a most hydraulically upstream point (L), the slope for the longest watercourse (S), the length from the concentration point along the longest watercourse to a point opposite the centroid (L_{ca}), and the basin roughness (K_n).

The first three parameters (L, L_{ca}, and S) are automatically computed in WMS 7.0. Some L_{ca}-values were adjusted based on engineering judgment since the WMS program could not correctly locate the point opposite the centroid for several sub-basins with meandering flow paths. In this case, half of L length was used for L_{ca}. The basin roughness is a composite Manning's *n* value for all the channels in the sub-basins. An appropriate basin roughness value is determined for each sub-basin based on field observation, areal photographs, soil maps, and land use maps. Lag time for each basin is calculated by using the formula in the Hydrology Manual (Sabol et al., 1995). The parameters for lag time calculation are summarized in Table D.2-5 (Appendix D.2).

Since Desert/Rangeland S-graphs are used for gently sloping natural areas, the K_n-value range is 0.02 to 0.03 as shown in Table 5-4 in the Hydrology Manual (Sabol et al., 1995). However, based upon our field survey, we determined a range of 0.04 to 0.06 according to the base material, vegetation, obstruction, and irregularity in the basins. The selection of K_n values is in agreement with studies conducted at similar surface geographic watersheds (Zhao, 2003).

As the Hydrology Manual (Sabol et al., 1995) notes, "some adjustment in K_n should be made for use with rainfalls of different magnitudes (frequencies)." We agree that K_n should be adjusted for storms with different return frequencies in the study since the study storm events vary from 10-year, 6-hour to the PMP 72-hour. However, no quantitative adjustment factors are available. Therefore, no adjustment is made to K_n -value for different storm events.

4.2.6.4 Channel Routing Path Parameters

The normal depth channel routing in HEC-1 is used to route runoff hydrographs through sub-basins. The routing paths for the drainage area can be found in Figure 4-2 (pocket).

The channel routing parameters include NSTPS (the number of sub-reaches to be divided for the routing reach by HEC-1), initial outflow, Manning's n-values for left overbank (LOB), main channel, and right overbank (ROB), reach length, bed slope for the routing reach, and an eight-point cross section (RX-RY data). The initial outflow is set as the initial inflow, which is usually zero (0) cfs, implying a dry channel when routing starts. The eight-point cross section data was obtained from field surveys (See Appendix C survey notes). The reach length and channel bed slope are automatically computed by WMS 7.0. The estimation of Manning's n-value is based on field trips, areal photo, land use maps, and soil maps using the procedures in USGS technical report: Estimated Manning's Roughness Coefficients for Stream Channels and Floodplains in Maricopa County, Arizona (USGS, 1991). NSTEPS values for routing reaches are calculated by an iterative process in WMS 7.0. All channel routing parameters are summarized in Tables D.3-1 through D.3-3 in Appendix D.3.

It is noted that several routing channels from field survey measurements are incapable of conveying peak discharges above the 100-year event (especially routing channels along the Sun Valley Parkway). This is because (1) there is really more than one routing channel per basin to convey the upstream discharges; (2) the routing channels are poorly-defined, and most of the flows are conveyed in the overbanks; and (3) the geological surface for the entire basin is flat. In those cases, the routing channel cross-sections are expanded according to the areal photos and DEM data.

4.3 Problems Encountered During the Study

4.3.1 Special Problems and Solutions

Several issues have already been proposed and discussed in the previous sections, such as unit hydrographs and basin roughness K_n-values. This section covers additional issues encountered during the study.

4.3.1.1 Time Interval for Rainfall

Previous hydrologic studies used a 30-minute interval for rainfall distributions, such as Alpha's (1996). In WMS 7.0, the Maricopa County Rainfall Distribution uses a 15-minute interval that gives better linear interpolations than the 30-minute interval. However, the peak discharge results from the 15-minute interval are typically 10% higher than the 30-minute interval provided that all other parameters are kept the same. A comparison was conducted for basins A through C as presented in Table D.6-1 (Appendix D.6). This 10% difference should be considered in verification or comparison with other studies and sources.

4.3.1.2 Phoenix Valley and Desert/Rangeland S-graphs

As mentioned in Section 4.2.6.2, Alpha (1996) used the Phoenix Valley S-Graph for natural desert rangelands because the Desert/Rangeland S-Graph was under development at that time. A comparison between Phoenix Valley and Desert S-Graphs was conducted for sub-basins A through C with all other parameters the same. The peak discharge results are summarized in Table D.6-2 (Appendix D.6). The Phoenix Valley S-Graph appears to provide a 0~23% greater discharge compared to the Desert/Rangeland S-Graph for the same basin.

4.3.1.3 Future Condition Assumptions

To comply with the sub-task 2.3.10.2 in the project Scope of Work, hydrologic analyses for Area 3 future conditions were performed. The following assumptions were used after discussion with the District.

1. Basin and sub-basin boundary delineations were kept the same as the existing conditions model. Basin delineation includes all the surface characteristics such as sub-basin boundaries, concentration points, and routing channels. Community Master Plans (CMP) within Buckeye Area 3 were reviewed. Most of the CMPs use the previous HEC-1 model in this area. For example, Tartesso used the same HEC-1 model developed by Alpha (1996). In their reports, they maintained the major natural channel and flow patterns and did not make significant changes to the topography. Based on these reports, we assume that the basin delineations for future conditions can be maintained the same as the existing condition.

2. Precipitation parameters are kept the same as the existing conditions. This includes rainfall amount for all storm events, areal reduction factors, and rainfall distribution patterns.

3. The future condition model will use the "land-use plan" provided by Town of Buckeye. Buckeye land use codes are different from the County's. The relationship between the two land use code systems is identified in Table 4-3. These changes are incorporated in GIS shape file and used to re-calculate the Green & Ampt parameters.

4. Retention is used for flood mitigation due to land development for future conditions. As stated in "Town of Buckeye Development Code (1996)", the Town does not allow any increase of peak discharge, runoff volume, and velocity of runoff for post-development; 100-year 2-hour onsite retention is encouraged. The retention volumes for each sub-basin are determined by a trial-and-error process. The existing and future condition 100-year 24-hour storm event HEC-1 models are simulated in a no routing option, and the difference of runoff volume generated from each sub-basin between the post- and pre-development is obtained, which is set as an initial retention volume. A second run of future condition HEC-1 model is then simulated with retention. The runoff volumes are compared to those of existing condition and retention volumes are adjusted until there is no increase of peak flows or volumes from each sub-basin. The final retention volume for each basin is presented in Table 4-4.

Table 4-3

Correlations between Buckeye and the County Land Use Codes

Buckeye Code	Description of Buckeye Code	FCDMC Code	Relation to County Code
CC	Community Center, to accommodate a variety of commerce and specialized development including commercial uses which comprise the central business district of the Town	C2	Central, general, office, intermediate commerce
GC	General Commerce, to accommodate general commercial and employment uses and compatible industrial uses to which public services are available	C2	Central, general, office, intermediate commerce
MR	Mixed Residential, to accommodate both single and multiple family residential development, historic residential neighborhoods and compatible commercial uses	MDR	Medium Density Residential, 6,000~12,000 sq. ft lot size
PC	Planned Community, to accommodate all land uses approved as part of a Community Master Plan, where specific uses, public services, densities, and design criteria have been identified and adopted	MDR	Developers proposed MDR
PR	Planned Residential, to accommodate all subdivided residential development to which public services are available	MDR	Developers proposed MDR
RR	Rural Residential, to accommodate low-density residential development in outlying areas where all public services may not be available.	VLDR	40,000 sq. ft and greater lot size
Special Use	To accommodate uses in natural hazard or floodplain areas or those under public ownership where development may not be possible.	NDR	Undeveloped desert rangeland
Undeveloped	Little topographic relief, slope < 5%, desert rangeland	NDR	Undeveloped desert rangeland

Table 4-4

Future Condition Retention Volumes for Buckeye Area 3

Sub-basin	Retention Volume [ac-ft]	Sub-basin	Retention Volume [ac-ft]
E1	53	L3	457
E2	118	M1	170
E3	152	M2	585
E4	190	N1	65
E5	79	N2	150
E6	100	O1	150
F1	133	P1	85
F2	130	Q1	72
F3	45	R1	24
G1	47	S1	61
H1	60	T1	103
H2	88	U1	47
J1	102	V1	23
J2	224	W1	0
K1	0	W2	130
K2	128	X1	0
K3	101	Y1	0
L1	0	Z1	58
L2	475		

Note:

1. **Retention volume zero (0) in the table indicates no future development is planned within that sub-basin.**
2. **Increase of peak discharge and runoff volume in the 100-year 24-hour storm event in the future condition is summarized in Table D.5-1, Appendix D. The runoff volume difference between the existing and future conditions was used as an initial retention volume;**
3. **The peak discharge and runoff volume for future conditions with final retention volume are summarized in Table D.5-2, Appendix D.**

4.3.2 Model Warning and Error Messages

No error messages are present in the computer model output files. There are several warning messages about the channel routing, including the following:

```
*** WARNING *** MODIFIED PULS ROUTING MAY BE NUMERICALLY UNSTABLE FOR
OUTFLOWS BETWEEN     2617. TO 3974.

                  THE ROUTED HYDROGRAPH SHOULD BE EXAMINED FOR OSCILLATIONS OR
OUTFLOWS GREATER THAN PEAK INFLOWS.

                  THIS CAN BE CORRECTED BY DECREASING THE TIME INTERVAL OR
INCREASING STORAGE (USE A LONGER REACH.)
```

A test was conducted by decreasing NSTEPS until the warning message disappeared. Since the "modified puls" routing method was not used, and there is less than 3% difference in peak discharge after the NSTEPS was changed, this type of warning is ignored.

4.4 Calibration

No calibration is performed because there is no gage station within the watershed. However, verification is performed (see Section 4.5.2, this report).

4.5 Final Results

4.5.1 Hydrologic Analysis Results

The results for Buckeye Area 3 hydrology were obtained using the 1998 revision of HEC-1 (version 4.1) embedded in WMS 7.0. PMP 6-hour and 72-hour storm events are shown in Table 4-5 for both existing and future conditions. The 6-hour PMF, which in this study is greater than the 72-hour PMF, generates flows that are approximately 500% of the 100-year storm.

4.5.2 Verification of Results

Previous studies in Area 3 were conducted by Alpha (1996) for 100-year storm events, and Dames & Moore (1990) for PMP storm events. The basin delineations and hydrologic parameters used in these studies as well as their results were compared to the parameters and results of the current study as verification.

The peak flows for the 100-year 6-hour storm event from this study were 10-15% higher than the Alpha study. Alpha determined a single areal reduction factor based on the entire watershed size, while this study based areal reduction factors on sub-basin size. The peak flows for the 100-year 24-hour storm event are very similar to each other for the same sub-basins (within ±10%).

The peak flows for PMP storm events from this study are 20% lower than that of the Dames & Moore study (1990). Dames & Moore used less detailed methods to delineate the watershed; and

used the SCS curve number method for rainfall-runoff conversion. The curve numbers determined in that study are relatively high compared to the Green and Ampt parameters used for this study. In addition, the rainfall precipitation amounts used in the Dames and Moore study were point values unmodified by the use of areal reduction factors.

Other verifications included the indirect methods presented in the Hydrology Manual (Sabol et al., 1995).

The results using indirect method No. 2 – USGS data for Arizona are presented in Figure 4-6. The LP3 regression curve and 75% tolerance limits from the USGS data are plotted along with the 100-year storm event peak discharges and trendline from this study. The results fit within the tolerance limits, and the trendline is very similar to the USGS's.

The second indirect method use Boughton and Malvick Envelope Curves, as presented in Figure 4-7. The Boughton and Malvick Envelope Curves are plotted along with the 100-year storm event peak discharge and trendline from this study. The results are all under the Boughton Envelope Curve, and are in agreement with the Malvick Envelope Curve.

The third indirect method of verification uses the USGS regional regression method (Thomas et al., 1995). Table 4-6 shows the USGS envelope curve data, 100-year regional data, and 100-year low-to-middle elevation peak discharges compared to basin size. Table 4-7 shows the peak discharges (100-year 6 & 24-hour) for all sub-basins in the Area 3 arranged in an ascending order of the sub-basin area. The comparison between Table 4-6 and Table 4-7 indicates the results from this study are relatively low compared with the regional 100-year discharges, but are all above the low-to-middle elevation values. The results are in agreement with those studies conducted in comparable areas (Zhao, 2003).

Table 4-5
Summary of Peak Discharges from Concentration Points along the Buckeye FRSs

			Existing Conditions		Future Conditions	
FRS	**CP**	**Area [mi^2]**	**PMP 6-HR**	**PMP 72-HR**	**PMP 6-HR**	**PMP 72-HR**
FRS#1	**E**	31.24	49,972	29,593	50,490	32,130
	G	1.12	4,114	2,389	4,202	2,429
	Z	0.66	2,775	1,616	2,957	1,719
	L	16.73	29,292	16,926	32,803	18,408
	M	13.12	23,749	13,252	25,268	14,225
	N	4.85	9,242	5,177	9,694	5,469
	O	3.09	6,584	3,719	7,023	3,946
	P	2.44	5,837	3,305	5,967	3,367
	Q	2.23	5,816	3,248	5,856	3,268
	R	0.28	1,716	1,036	1,795	1,081
FRS#2	**S**	1.41	9,642	4,479	9,757	4,539
	T	3.04	15,022	6,656	15,109	6,696
	U	1.37	8,841	4,033	8,898	4,059
FRS#3	**V**	0.66	3,462	1,618	3,483	1,628
	W	6.74	28,456	12,909	28,604	12,995
	X	0.79	4,163	1,994	4,164	1,994
	Y	0.58	4,868	2,556	4,867	2,556

Discharge unit: cubic foot/second (cfs)

Figure 4-6

Indirect Method Verification - Buckeye Area 3 Study and USGS LP3 Analysis

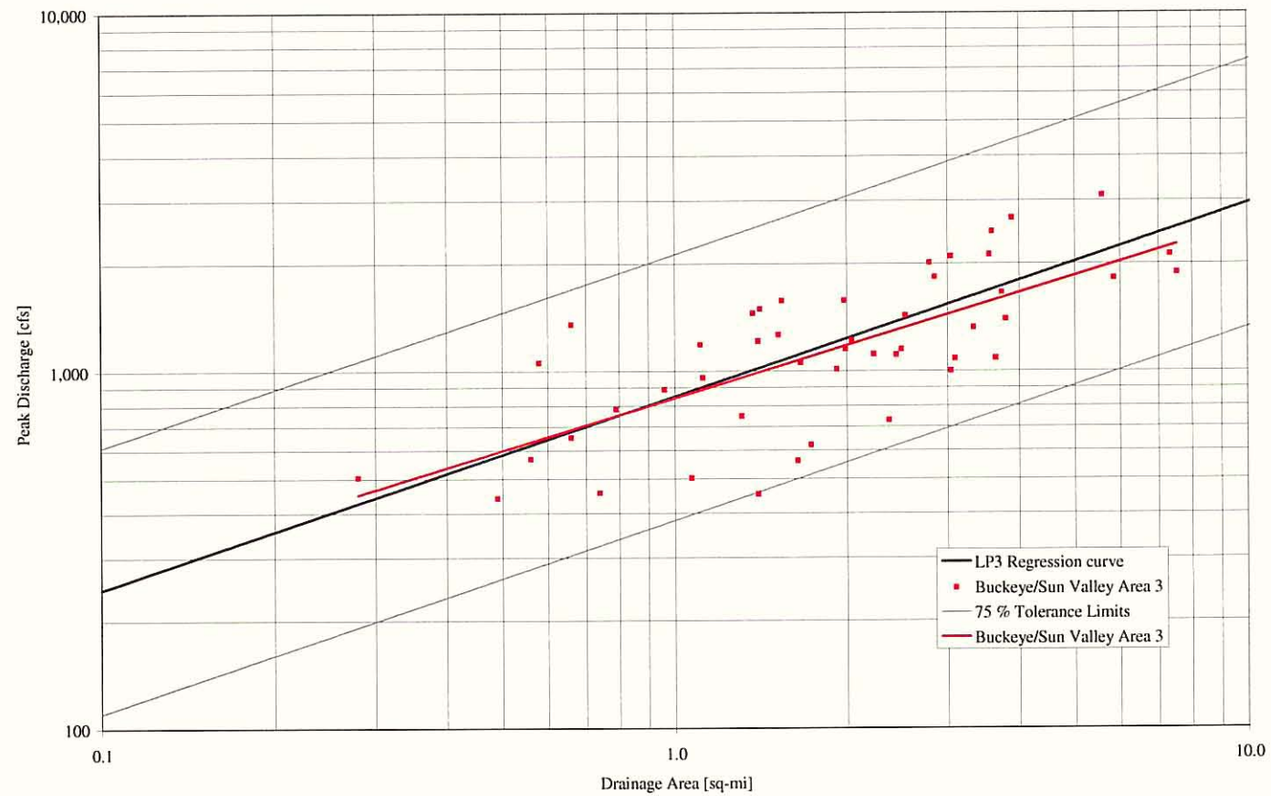


Figure 4-7

Indirect Method Verification - Buckeye Area 3 Study and Boughton and Malvick Envelope Curves

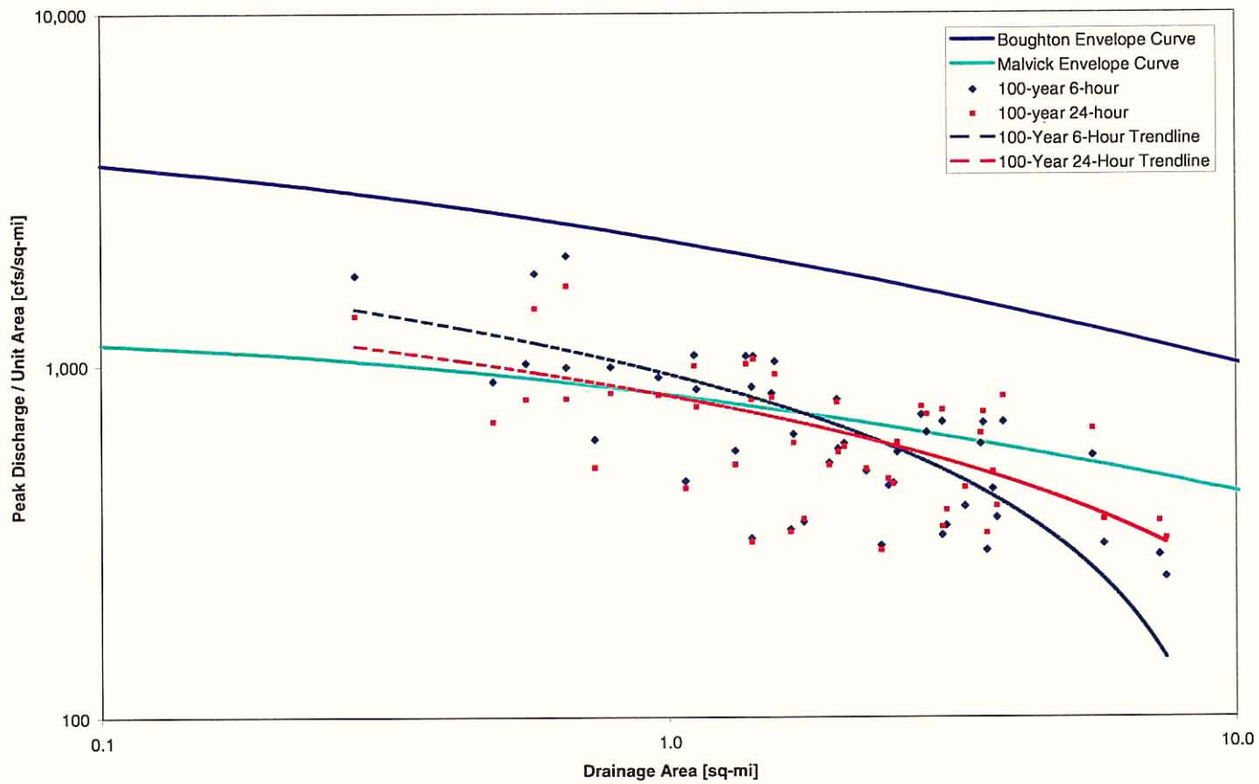


Table 4-7

USGS Regression Data (Digitized from Figure 41 in Thomas et al., 1995)

Drainage Area [sq-mi]	Envelope [cfs]	Regional [cfs]	Low-to-Middle Elevation [cfs]
0.1	300	182	110
0.2	700	308	156
0.3	1,162	419	192
0.4	1,600	522	222
0.5	2,028	618	249
0.6	2,500	710	273
0.7	3,000	798	296
0.8	3,500	884	317
0.9	3,887	967	336
1	4,403	1,047	355
2	7,878	1,678	505
3	10,676	2,210	621
4	13,809	2,688	720
5	16,047	3,128	806

Table 4-8

Area 3 Sub-Basin Existing 100-year Peak Flows

Sub-Basin	Area [sq-mi]	6-Hr Peak [cfs]	24-Hr Peak [cfs]
R1	0.28	506	387
Y1	0.58	1,077	837
V1	0.66	664	526
Z1	0.66	694	558
F3	0.74	452	374
X1	0.79	787	645
E1	0.96	894	790
H1	1.07	497	472
G1	1.12	928	820
U1	1.37	1,496	1,365
H2	1.4	446	432
S1	1.41	1,542	1,449
N1	1.52	1,253	1,211
K1	1.54	1,563	1,432
K3	1.64	520	507
J1	1.73	626	633
E2	1.98	1,587	1,548
E6	2.04	1,227	1,194
Q1	2.23	1,107	1,111
K2	2.37	721	694
P1	2.44	1,041	1,073
E3	2.49	1,147	1,129
F2	2.53	1,426	1,506
W2	2.85	1,878	2,042
E4	3.04	923	966
T1	3.04	2,111	2,151
O1	3.09	1,057	1,163
N2	3.33	1,193	1,331
L1	3.55	2,127	2,267
J2	3.64	1,095	1,213
W1	3.89	2,623	2,976
M1	5.59	3,143	3,717
L3	5.85	1,832	2,129
L2	7.33	2,135	2,641
M2	7.53	1,812	2,304

5 Hydraulics

5.1 Method Description

The Buckeye/Sun Valley watershed is in west-central Maricopa County, Arizona, about 30 miles west of central metropolitan Phoenix. The watershed drains south and west from the western slopes of the White Tank Mountains.

The Buckeye FRSs were designed and constructed by the SCS in the 1970s and have a combined contributing drainage area of 90 mi^2. The three FRSs (#1-3) were designed to function as a single system with the detained flows cascading west to the eventual outfall to the Hassayampa River. The structures provided protection to downstream agricultural lands and the Town of Buckeye from storms up to the one percent exceedance level. Ungated low level outlets serve as principal spillways releasing runoff to the west. Free surface emergency spillways were provided for each FRS to control discharge for storms in excess of a storm event equivalent to "74% of the 100-year storm plus 26% of the probable maximum flood (PMF)". The dam crest, termed as "emergency spillway freeboard", was designed to contain the PMF without overtopping (NRCS, 1985; Arizona Water Commission, 1979). The principal outlet discharges are routed to the FRSs through Buckeye Floodways #2 and #3. Emergency spillway discharges do not cascade to other FRSs but flow generally to the south toward the Gila River floodplain (Figure 5-1). Selected engineering design data for the three FRSs are presented in Table 5-1 (Arizona Water Commission, 1979).

The Buckeye FRS system was inspected and assessed by the Corps of Engineers (Arizona Water Commission, 1979) and Dames & Moore (1990) following construction. Both studies included hydrologic analyses for the 100-year and probable maximum precipitation (PMP) events using the HEC-1 level pool routing method, and concluded all three FRSs would not be safe to convey the PMF.

The level pool routing method in HEC-1 was used for FRS design and assessment; however, it cannot model the spatial distribution of flow hydrographs. In addition, it cannot account for many hydraulic conditions including backwater effects, hydraulic structures, and unsteady effects (Fread, et al., 1988).

In terms of risk assessment, quantitative evaluation of water surface elevation (WSEL) corresponding to precipitation frequency is critical to the operator and downstream populations at risk. In this study, unsteady models were selected to develop stage-frequency relationships for the FRSs. The US Army Corps of Engineers HEC-RAS (version 3.1.1, May 2003) computer program was used to establish the models. Hydrologic responses from PMP 6/72-hour storm events were obtained and input in the unsteady models. The starting WSEL was simulated close to zero as there is no base flow in the FRSs. However, it is required to input an initial flow to start the unsteady models. For this analysis, a base flow of 20-50 cfs was used.

Table 5-1

The Buckeye FRSs Engineering Design Data (NRCS, 1974)**

Item	Unit	Buckeye FRS Structures FRS#1	FRS#2	FRS#3
Embankment				
Length	miles	7.0	2.3	3.0
Maximum Height	feet	48.0	26.0	34.0
Crest Elevation§	feet	1,088.0*	1,117.0	1,170.0
Crest Elevation‡	feet	1,089.5*	1,118.6	1,171.5
Principal Spillway				
Conduit Diameter	inches	60.0	48.0	30.0
Emergency Spillway				
Crest Width	feet	800.0	350.0	400.0
Crest Elevation§	feet	1,079.8	1,111.2	1,163.2
Crest Elevation‡	feet	1,081.8	1,113.2	1,165.2
Reservoir				
Surface Area				
at E. Spillway Crest	acres	1,137.0	150.0	180.0
at Dam Crest	acres	1,952.0	235.0	335.0
Storage Volume				
at E. Spillway Crest	acre feet	8,200.0	780.0	920.0
at Dam Crest	acre feet	19,024.0	1,920.0	2,786.0

*Buckeye FRS #1 embankment crest includes a 5,580-foot-long level section at elevation 1088.0 feet (NGVD29), a 31,500-foot-long level section at elevation 1089.5 feet and a 600-foot-long sloping transition section between the two level sections.

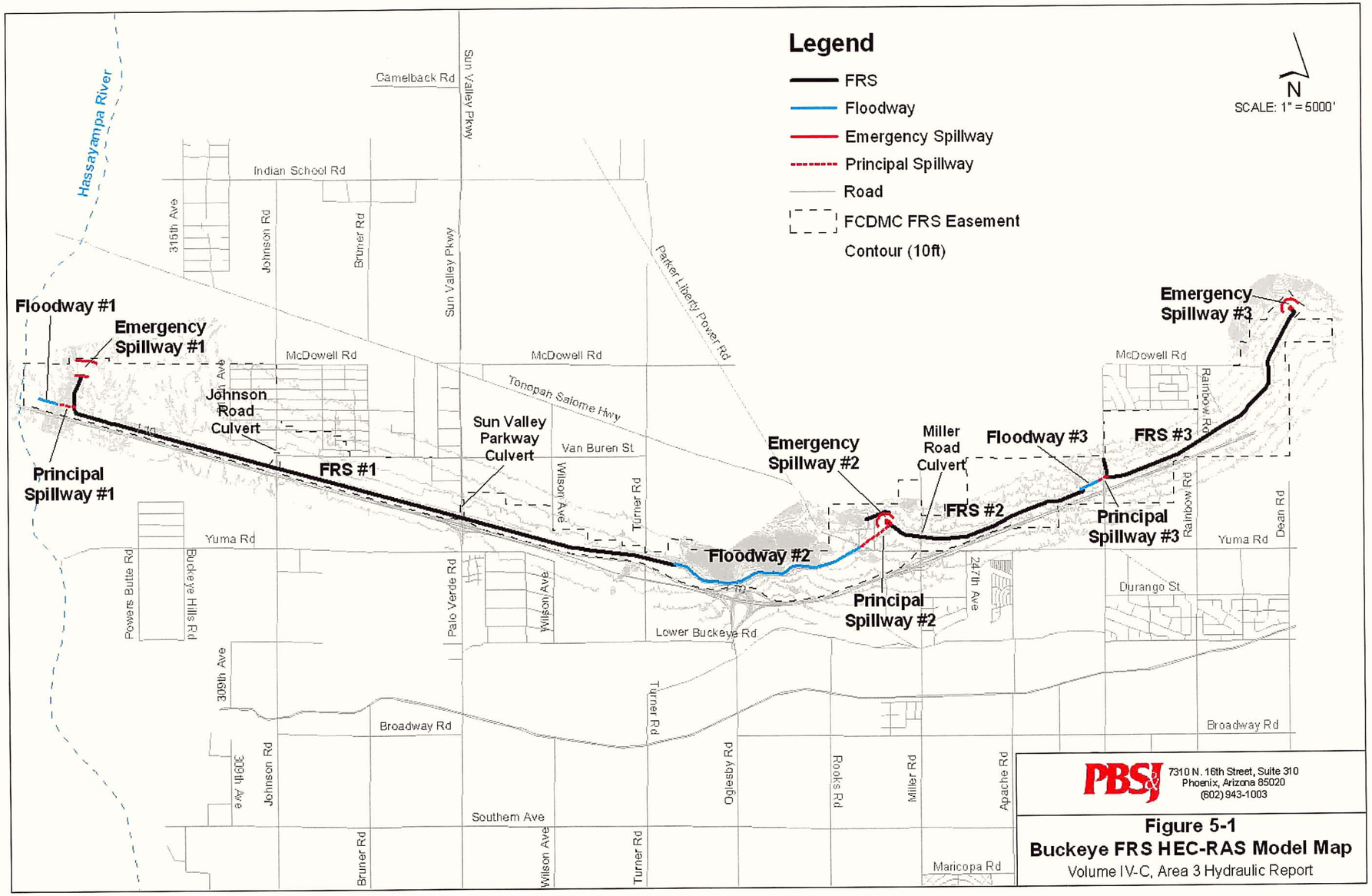
§ Vertical Datum NGVD29

‡ Vertical Datum NAVD88

5.2 Work Study Maps

Initially, PBS&J tried to set up a single hydraulic model combining all three FRSs; however, the model was unstable due to its length and complexity, and it would have been more difficult to update the results. Consequently, a model was developed for each individual FRS based on the physical characteristics of the structures.

A reduced-scale general HEC-RAS model overview map of the Buckeye FRS system is shown in Figure 5-1 with each FRS delineation and key features identified. In addition, full scale (1"=200') work study maps including hydraulic baseline, property line, PMF boundary, and cross section information are provided in the pockets (Figure 5-2 to 5-17).



PBSJ 7310 N. 16th Street, Suite 310 Phoenix, Arizona 85020 (602) 943-1003

Figure 5-1
Buckeye FRS HEC-RAS Model Map
Volume IV-C, Area 3 Hydraulic Report

5.3 Parameter Estimation

5.3.1 Roughness Coefficients

Elements to determine roughness coefficients include base materials, surface irregularities, obstruction, and vegetation by following "Estimated Manning's Roughness Coefficients for Stream Channels and Flood Plains in Maricopa County, Arizona" (USGS, 1991). The Buckeye FRSs were constructed in 1970's and have been maintained by the District since then. Field investigations indicate that the left overbanks (LOB) from the top of the dam crest to the edge of main channel are covered by flexible turf grasses and underlying firm soil. The main channel has sparse vegetative cover and a combination of firm soil, coarse sand, and fine gravels. The right overbanks (ROB) from the main channel to the edge of the foothills have medium scattered brushes and weeds (Appendix C).

A summary of the roughness coefficients used in the unsteady models is presented in Tables 5-2a and b.

Table 5-2a

Roughness Coefficients for Buckeye FRS #2 & 3

	LOB		Main Channel		ROB	
	n	**Description**	**n**	**Description**	**n**	**Description**
n_b	0.020	Firm soil	0.025	Firm soil, coarse sand; and fine gravel	0.020	Firm soil
n_1	0.005	Minor	0.005	Minor	0.005	Minor
n_2	0.004	Negligible	0.004	Negligible	0.004	Negligible
n_3	0.011	Small weeds	0.001	Negligible to small	0.016	Medium brush, weeds
n total	0.040		0.035		0.045	

Table 5-2b

Roughness Coefficients for Buckeye FRS #1

	LOB		Main Channel		ROB	
	n	Description	n	Description	n	Description
n_b	0.020	Firm soil	0.025	Firm soil, coarse sand; and fine gravel	0.020	Firm soil
n_1	0.000	Minor	0.000	Minor	0.005	Minor
n_2	0.000	Negligible	0.000	Negligible	0.004	Negligible
n_3	0.010	Small weeds	0.005	Negligible to small	0.016	Medium brush, weeds
n total	0.030		0.030		0.045	

Where $n = n_b + n_1 + n_2 + n_3$ (Equation 1)

n_b = base value of n for a straight uniform channel,

n_1 = value for surface irregularities,

n_2 = value for obstruction, and

n_3 = value for vegetation.

5.3.2 Expansion and Contraction Coefficients

Expansion and contraction coefficients were applied where flow constrictions or expansions were present or where flow approaches and leaves structures. Expansion and contraction coefficients used 0.3 and 0.1 for uniform conditions, respectively. Entrance and exit loss coefficients used 0.5 and 1.0 for culverts, respectively.

5.4 Cross Section Description

WSELs were computed at hydraulic sections along each FRS. The sections were developed using GIS from mapping data prepared by Kenney Aerial Mapping, Inc. in 2003, as-built plans, and field reconnaissance. Sections were generally spaced at 200-foot intervals, and more cross sections were added where structures (roadway, culverts) were encountered along the FRSs. Hydraulic sections were defined by data points oriented from left to right and looking downstream (southwesterly). Transverse stationing increases left to right.

Longitudinal "river mile" stationing increases upstream and reflects the distance in feet between cross sections. The downstream limit of the FRS #1 reach is at the outfall to the Hassayampa River; the downstream limit of the FRS #2 reach is at the end of Floodway #2; and the downstream limit of the FRS #3 reach is at the end of Floodway #3. Each downstream limit river station begins at 0+00.

Vertical control for cross sections and longitudinal profiles is based on the mapping data, which utilizes the North American Vertical Datum of 1988 (NAVD 88). Part of the data for the

culverts and principal spillways were obtained from the 1974 as-built plans, which were based on the National Geodetic Vertical Datum of 1929 (NGVD 29). They were converted to NAVD 88 prior to use.

The floodwater retarding structures were designed to include aerated sediment storage. This ADMS will include an estimate of watershed sediment yield, but it is not reasonable to assume that all of this sediment will actually migrate all the way to the floodpool. Data from the District Dam Operation and Safety Monitoring Program indicate minimal sediments have accumulated within the FRS floodpool after 30 years' operation. Therefore, the unsteady model used the ground terrain data.

5.5 Modeling Considerations

5.5.1 Hydraulic Jump and Drop Analysis

N/A

5.5.2 Bridges and Culverts

There are three roadways crossing the FRSs (Figure 5-1). A culvert was placed for flow conveyance underneath each roadway. HEC-RAS input data include culvert type, length, upstream and downstream invert elevations, deck width, entrance and exit loss coefficients and weir coefficients. Data for the culverts were obtained from the GIS mapping, field survey, and/or the as-built drawings (NRCS, 1974). Typically, there was no adjustment in culvert geometry to account for potential debris clogging at any culvert. Culvert crossings vary in size, shape, length, and number of barrels. A roughness coefficient of 0.013 was used for all culverts. A detailed hydraulic calculation is provided in the HEC-RAS printout in Appendix E.

The principal spillway of each FRS is also modeled as a culvert, rather than using a rating curve. A thorough discussion of the principal spillway modeling and rating curves is addressed in Section 5.7.1 this report.

5.5.3 Dam Crest and Emergency Spillway

To quantify the extent of overtopping of the FRSs during various storm events, the FRS emergency spillways and the dam crests were modeled as broad-crested weirs. The HEC-RAS program does not allow overlapping of structures in the model. In this case, where a roadway crossed the FRS, the broad-crested weir used to model the dam crest was divided into several segmental weirs. The flow discharge over the weirs was calculated by following the weir equation:

$$Q=CLH^{1.5} \qquad \text{(Equation 2)}$$

Where, Q = flow over the weir (cfs),

C = weir coefficient of discharge,

L = length of the weir perpendicular to the flow (ft), and

H = difference between the reservoir mean water surface elevation and the crest elevation for the weir (ft)

The weir coefficient of discharges for each FRS was taken from Dames & Moore's dam break study (1990).

The geometric data for the dam crests and emergency spillways were obtained from the following sources.

- Field surveys performed by others for the District for an ongoing dam safety assessment (Appendix C).
- New 2-ft contour mapping
- As-built drawings (NRCS, 1974).

5.5.4 Islands and Flow Splits

N/A

5.5.5 Ineffective Flow Areas

N/A

5.5.6 Supercritical Flow

N/A

5.6 Floodway Modeling

N/A

5.7 Problems Encountered During the Study

5.7.1 Special Problems and Solutions

Several technical problems were encountered during development and refinement of the unsteady model. The models were initially developed as simply as possible. When the model achieved a stable condition, more features such as roadways, culverts, principal and emergency spillways, and dam crests were added to the model.

During development of the unsteady model, it became clear that it is very sensitive to the sinuosity of the hydraulic reaches, the complexity of the hydraulic structures, and especially to the computational intervals. The following sections discuss the major constraints encountered and the approach taken for each.

5.7.1.1 Modeling of Roadways Transverse to the FRS

There are three significant existing roads, which cross the dam structures. These are Miller Road, which crosses FRS #2, and the Sun Valley Parkway and Johnson Road crossings of FRS #1. The impacts of roadways were ignored in the previous study (Dames & Moore, 1990). It had been assumed that the impact of roadways would be negligible since the volume of roadway fill was minimal compared to the storage volume of the dams.

Because of the concentration points distributed on both sides of the intersecting roadway, it seemed possible that the flow could reverse direction over the roadway surface in a severe storm event. Reversal of flow direction over the roadway surface was indeed observed in the HEC-RAS model, and WSELs on each side of the roadway differed due to hydraulic losses as the water overtopped the roadway. The difference between WSELs on both sides significantly reached up to 3 ft at the PMF peak discharges and could not be reasonably ignored. These transverse roadways dramatically changed the hydraulic profiles as designed by the NRCS in 1974.

5.7.1.2 Modeling of the Emergency Spillway and Dam Crest for Overtopping

Two alternatives were considered for modeling the extent of overtopping. The first method assumes that flows are discharged only through the primary and emergency spillways, and water is "stacked" as high as needed to provide the hydraulic head required. This option calculates the highest WSEL and provides the worst-case scenario. This method was used for the analysis in the Spook Hill FRS PMF Hydraulics Study (DMJM, 2002).

The other option includes overtopping discharges. The emergency spillways and the dam crests are treated as lateral broad-crested weirs. Outflow over these weirs only occurs when the WSEL in the FRS is above the crest elevations. This option directly calculates the amount of flow diverted through different areas of the dam spillways or crest. This alternative was used for this study because the District is updating the inundation mapping downstream of each emergency spillway.

5.7.1.3 Rating Curves versus Direct Calculation of the Spillway

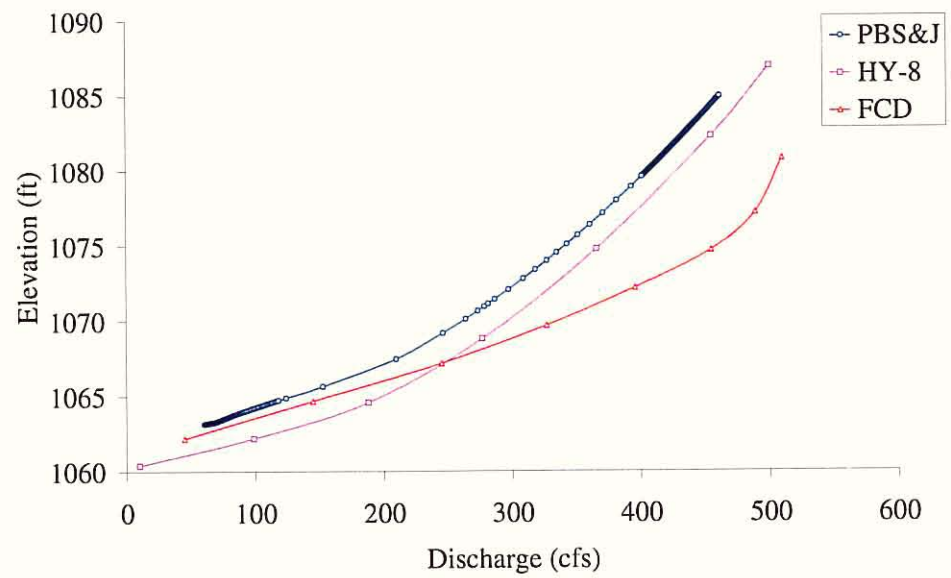
Initially, we planned to use rating curves for the spillways and culverts. The principal outlet rating curves were made available by the District for that purpose. However, the final model did not use the provided rating curves for the following reasons:

- All other structures in the unsteady model are simulated with data from field survey and as-built plans. Using physically based data for culverts rather than rating curves provided a more consistent approach.
- The FRS principal spillways are 600-1,200 feet in length, with more than 20 feet of fall. The rating curve assumes the hydraulic stage-discharge relationships at the upstream and downstream ends of the principal spillway are identical, which does not agree with either the field survey or the modeling results.
- Rating curve cannot consider backwater effects (Figure 5-18).

The stage-discharge relationship at the upstream end of the principal spillway developed from the unsteady model was compared to the rating curve provided by the District and the results from a separate program HY-8 (FHWA, 1995) in Figure 5-18. They are in agreement with each other in general. However, the unsteady model counts dynamic WSEL and discharge, and generally gives a lower discharge for the same stage.

Figure 5-18

Rating Curves from the District and the Unsteady Model by PBS&J



5.7.1.4 Low Flow Conditions

This issue was identified by the District reviewer, Mr. Joe Rumann, Senior Hydrologist. He observed energy grade oscillations at low flow in the FRS#2 Unsteady Model, especially in Floodway No. 3, upstream of FRS #2. When the discharge in the channel is less than 400 cubic feet per second (cfs), the velocity exhibits extreme oscillation, going from zero to 120 feet per second (fps) with associated energy grade oscillations.

PBS&J re-checked the parameters of the FRS unsteady models, and found this oscillation occurred in all FRSs at all storm events. Furthermore, a literature research indicated that HEC-RAS does not handle low flow situations well. Several approaches were tested to mitigate this oscillation, based on discussions with Mr. Rumann. Below is a summary addressing this issue.

1. It appeared that the relatively steep slope of the floodway might be part of the problem, and so allowing the model to run in supercritical mode might mitigate the problem.

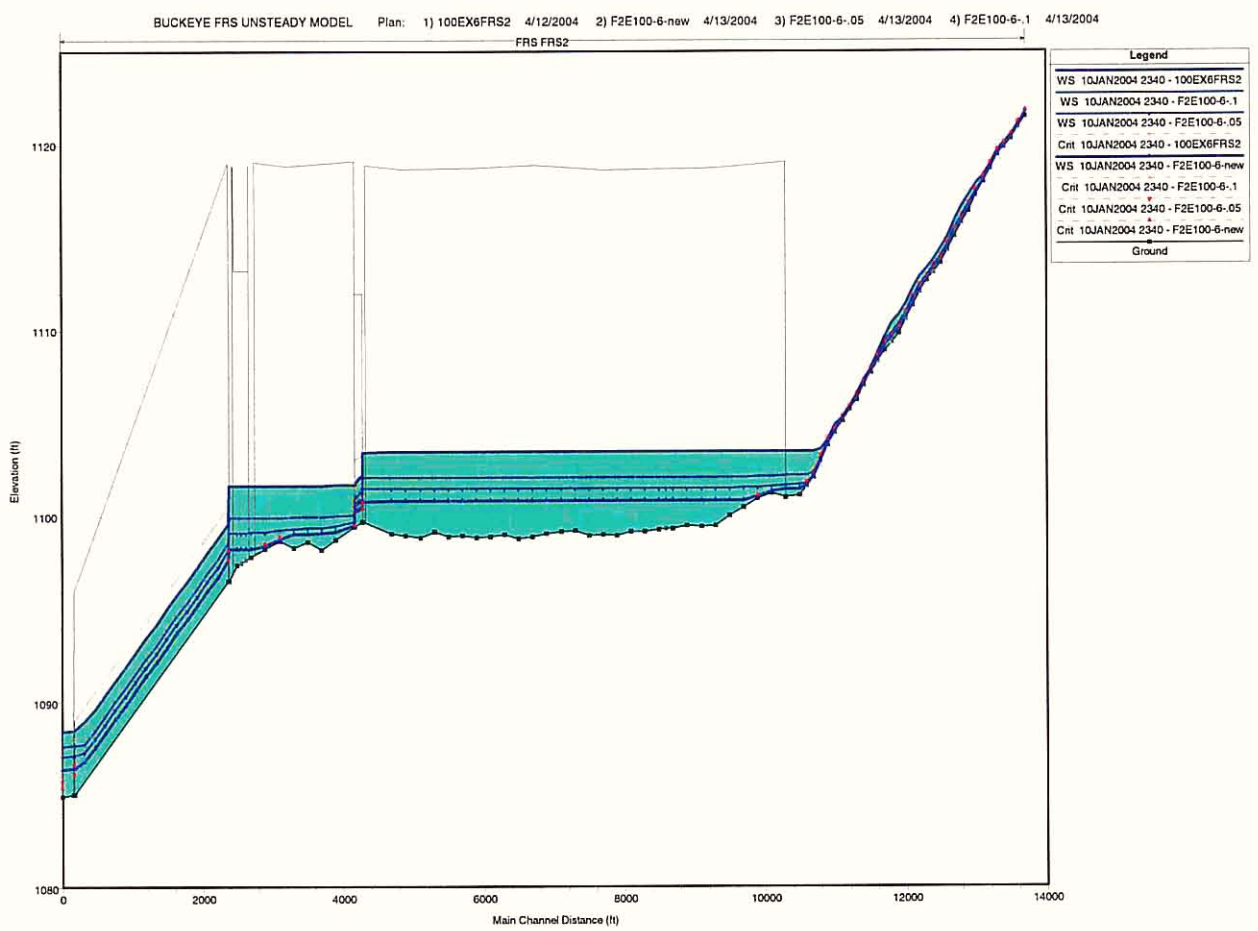
However, the oscillation still remained even in a mixed flow regime. Therefore, the subcritical mode was used for all the unsteady models.

2. Increasing the initial flow mitigates the oscillation but does not eliminate it. There is no base flow in the FRSs. However, the unsteady model requires an initial flow to start the calculation. To solve this dilemma, a small amount of initial flow was used (20-50cfs). We increased the initial flow up to 100 cfs and less oscillation was observed, but it still existed. Initial flow greater than 100 cfs was considered to distort the overall results.

3. Increasing the water surface calculation tolerance limit (TL) can eliminate the oscillation, but this also decreases accuracy and sensitivity of the model. Different TLs (0.02, 0.05, 0.1, and 0.2 ft) were used to test the results. All of them gave the same maximum WSEL, However, the oscillation did not disappear until TL was set at 0.2 ft. The results indicated at low flow, the higher the TL used, the higher the WSEL was observed. At the end of the simulation (the 10th day) for the FRS#2 unsteady model in the 100-year, 6-hour storm event, the results from TL=0.2 ft is 1.5 ft higher than that from TL=0.02ft (Figure 5-19).

4. A slot channel in the floodway might eliminate the oscillation. References about solutions to low flow oscillation in the HEC-RAS model suggest that placement of an imaginary slot channel two to three feet deep and six inches or less wide at the bottom of the main channel would increase the stability of the model without affecting the final results. In this case, the low flow will be contained within the slot channel; therefore, no oscillation occurs. Since the results of the low flow condition are not considered a major focus in this study and substantial additional effort would be required, it was decided that this alternative would not be investigated as part of this study

Figure 5-19
Impact of Tolerance Limits to Water Surface Calculations



5.7.1.5 Floodway Overtopping during PMF events

The floodways were designed and constructed to convey primary spillway releases (NRCS, 1974), and are adequate for this purpose. However, when the flood pool starts to fill up during a severe storm event, WSEL along the associated floodways exceeds the overbank limits due to backwater effects. This occurs for the PMP 6 & 72 hour storm events. The max WSEL is between one and two feet higher than the bank elevation of Floodway #2 (between FRS #1 and FRS #2) for more than 2 miles, and one foot higher than the bank elevation of Floodway #3 (between FRS #2 and FRS #3) for a quarter mile. As a result the model displays a warning message "*The cross-section end points need to be extended vertically for the computed water surface*" for the floodways. Under this scenario, it will act as a secondary spillway and cause inundations. The model was modified by setting the left overbank of these overtopped floodway segments as lateral broad-crest weirs, and rerun for the PMF event.

For the FRS#2, two scenarios were simulated. One is with Miller Rd., and the other is without. Miller Rd is a dirt road, and could be washed out during a PMF event. The results are shown in

Table 5-3. The original model results were presented as a comparison. The floodway discharge does not affect the emergency spillway hydraulics characteristics at all. However, the dam crest overtopping discharge is lessened and the maximum depth on the dam crest decreases by 0.2 ft. When Miller Rd is assumed washed out, the floodway discharge is reduced by half, and the dam crest overtopping is significantly decreased from 6,856 cfs to 336 cfs. However, the emergency spillway discharge is doubled. The above results are valid since the accuracy of the model is ±0.05 cfs for discharge and ±0.02 ft for WSEL. For the FRS#1, the results showed the same trend as to the FRS#2 (Table 5-4).

Table 5-3

Floodway discharge at the PMF for the FRS#2

		Peak Discharge (cfs)	Max Depth on Weir (ft)
No Floodway Discharge	Floodway #3	0	0.00
	Dam Crest upstream of Miller Rd	10,970	0.91
	Dam Crest downstream of Miller Rd	0	0.00
	Emergency Spillway #2	6,856	3.71
Floodway Discharge with Miller Rd	Floodway #3	4,624	1.86
	Dam Crest upstream of Miller Rd	6,904	0.71
	Dam Crest downstream of Miller Rd	0	0.00
	Emergency Spillway #2	6,856	3.71
Floodway Discharge with Miller Rd Washed Away	Floodway #3	2,258	1.40
	Dam Crest upstream of Miller Rd	336	0.19
	Dam Crest downstream of Miller Rd	0	0.00
	Emergency Spillway #2	12,143	5.40

Table 5-4

Floodway discharge at the PMF for the FRS#1

		Peak Discharge (cfs)	Max Depth on Weir (ft)
No Floodway Discharge	Floodway #2	0	0.00
	Dam Crest upstream of Sun Valley PKWY	25,519	1.30
	Dam Crest downstream of Sun Valley PKWY	0	0.00
	Emergency Spillway #1	18,887	4.23
Floodway Discharge	Floodway #2	8,943	1.88
	Dam Crest upstream of Sun Valley PKWY	15,723	1.11
	Dam Crest downstream of Sun Valley PKWY	0	0.00
	Emergency Spillway #1	18,887	4.23

5.7.2 Modeling Warning and Error Messages

No Error Messages were noted in the model output.

There are several warning messages presented in the model result window.

1. Warning: The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4. This may indicate the need for additional cross sections.

   This type of warning is caused by the rapid change of inflow discharges in the hydrograph, especially under future conditions. Spacing of the cross sections has already been reduced to less than 200 feet. When additional cross sections were applied, the warnings were reduced, but not eliminated. A comparison of the results from 100-foot spacing and 200-foot spacing for the same FRS reach indicated that the WSEL difference was less than 0.02 foot. Therefore, the warning was ignored.

2. Warning: The cross-section end points need to be extended vertically for the computed water surface.

   There are floodways connecting the FRS, which are inundated by the flood pool of the FRS dams. This highlighted an issue for the District that the area around the floodway potentially could be flooded in a severe storm event. Additional analysis and floodway overtopping issues were addressed in Section 5.7.1.5.

5.8 Calibration

No calibration was performed because there is no available gage data in the FRSs. However, verification was performed (see Section 5.9.2, this report).

5.9 Final Results

5.9.1 Hydraulic Analysis Results

Overall 48 unsteady hydraulic models were developed for the three FRSs and eight storm events including both existing and future conditions. The models assume the Buckeye FRS system will maintain its structural integrity. Dramatic changes of flow properties such as water elevation, discharge, flow direction, and velocity with the temporal and spatial variations in the FRSs were observed. An additional benefit of the HEC-RAS program is that results of the unsteady model can be presented graphically, illustrating the dynamic hydraulics of the FRSs. Detailed results in the form of avi clips are presented in the attached DVD.

5.9.1.1 Hydraulic Responses up to 500-Year Storm events

The maximum WSEL in each FRS up to the 500-year storm event (existing condition) are presented in Table 5-5. The WSEL under future conditions is 0.2-0.5 ft less due to attenuation of

the peak flow and runoff volumes by retention basins (Volume III-C: Area 3 Hydrology Report). Contrary to the findings of previous studies performed on the structures (Dames & Moore, 1990), the HEC-RAS analysis indicates that the FRS system can safely contain and convey runoff for storms up to the 500-year event without overtopping the emergency spillways.

There are at least two reasons for this result. First, the Dames & Moore study assumed that all three principal spillways in the Buckeye FRS system were plugged. Second, the level pool routing method used in its study assumed the inflow hydrograph in each FRS is a simple addition of multiple inflow hydrographs from the concentration points along the FRS, and no spatial variations were considered.

Table 5-5

Maximum Water Surface Elevations for up to the 500-Year Storm Event (Existing Conditions)

	Water Surface Elevation (ft)		
Storm Event	FRS#1	FRS#2	FRS#3
10-year 6-hour	1075.53	1105.44	1155.77
10-year 24-hour	1074.20	1105.69	1156.59
100-year 6-hour	1080.06	1110.23	1160.47
100-year 24-hour	1078.31	1109.10	1159.47
200-year 24-hour	1079.43	1109.92	1160.24
500-year 24-hour	1081.03	1110.88	1161.29
Emergency Spillway Elevation	1081.97	1113.23	1165.23

5.9.1.2 Hydraulic Response to PMP Events

The three FRSs respond differently to PMP events. The results for all FRS cross sections and culverts/weirs for the PMF are shown in Table 5-6 and Table 5-7, respectively. For future conditions, there is less than 0.2 ft increase of the WSEL in the post-development scenario. In this study, the PMP 6-hour storm is considered to generate the PMF. River stations were assigned a letter after the number to represent concentration points along the FRSs. River stations with an asterisk (*) are interpolated cross sections generated by the HEC-RAS program. Negative values for total discharge and average velocity indicate reverse flow direction.

The maximum WSELs, maximum dam crest and emergency spillway overtopping flows, duration, and water depths for each FRS are presented in Table 5-8. The emergency spillways are overtopped for all three structures; however, only FRS#2 and #1 dam crests are overtopped. The 6-hour PMP storm event creates higher discharges and water depths, but the 72-hour PMP storm event causes 10-60% longer overtopping durations on each emergency spillway.

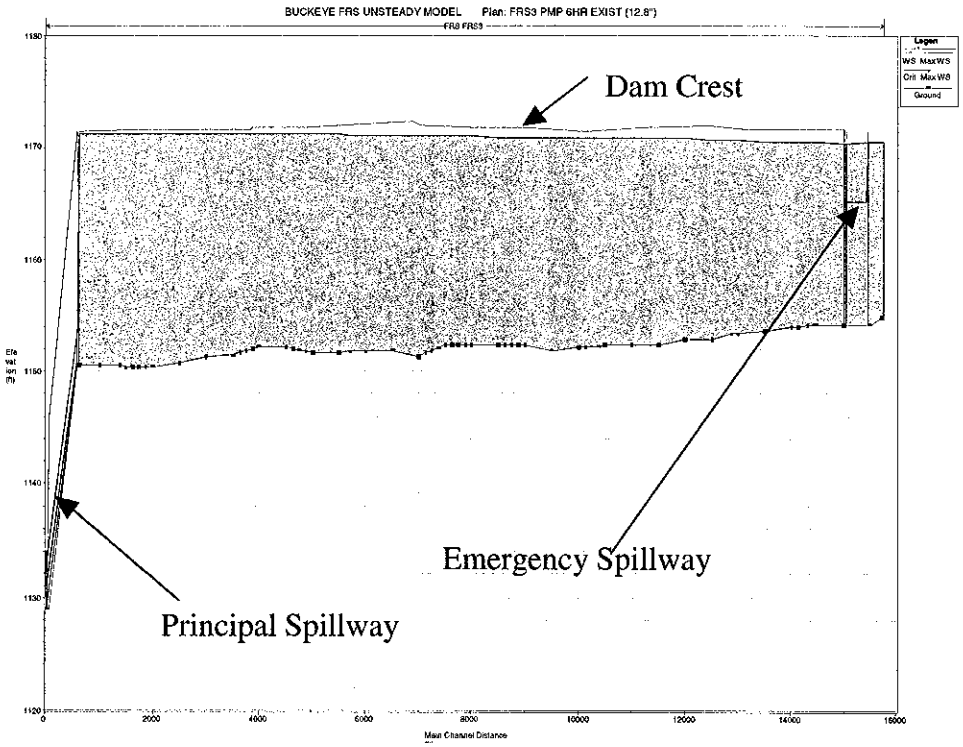
For comparison purpose, the results of the Dames & Moore study (1990) are also presented in Table 5-8. In its analysis, level pool routing was used for FRS stage computations. Besides the routing assumptions previously stated, Dames & Moore further assumed (1) the emergency

spillway capacity was fully utilized, (2) at any given moment, there is only one WSEL at any location within the entire FRS (level pool routing), and (3) impacts of roadways and culverts crossing the FRS were ignored.

For FRS#3, the unsteady hydraulic models consider spatial and temporal variations, and thus give lower maximum WSELs than Dames & Moore's (Table 5-8). As a result, no overtopping on the dam crest is observed in FRS#3 for either the 6-hour or 72-hour PMP events. Principal spillway discharge contributes to lower maximum WSEL in the unsteady model as well. Even with no roadway crossing in FRS#3, there is a 1.2 ft drawdown in the WSEL approaching the emergency spillway located at the upstream end (Figure 5-20).

The maximum WSELs in the Dames & Moore study for FRS#1 and FRS#2 are nearly the same as or even slightly less than the unsteady model results (Table 5-8). The explanation can be found in its assumptions. There are roadways crossing FRS#1 and FRS#2 (Figure 5-1). In order to fully utilize the emergency spillway capacity, there should be enough momentum to drive the flow up to the appropriate stage. The roadways have already consumed a certain amount of momentum, and there is not enough left to drive the flow over the dam crest downstream of the roadway. While the upstream flow overcomes the resistance of roadways and reaches the emergency spillway (i.e., 4.5 miles from the upstream roadway to the emergency spillway for FRS#1), the inflow hydrographs have already begun to recede. The lower momentum also substantially increases the durations of overtopping of dam crests and emergency spillways compared to the results from level pool routing (Table 5-8).

Figure 5-20
Maximum Water Surface Drawdown toward Emergency Spillway for FRS#3 in the 6-hour PMP Event



5.9.2 Verification of Results

As required by the District, level pool routing analyses were conducted for dual purposes – to update the Dames & Moore (1990) study and to compare to the unsteady hydraulic model results.

Unlike the Dames & Moore study, the current level pool routing analyses didn't assume the principal spillway plugged. The rating curves used for principal spillways were provided by the District. Vertical control was based on the NAVD 88 datum.

A summary of the level pool routing results is presented in Table 5-9, and a comparison of the level pool results and the unsteady model results is presented in Table 5-10. Results from both analyses are comparable. However, the interpretations may be different:

- Without consideration of structural impacts, the unsteady model gives lower maximum WSEL than level pool routing does. As shown in Table 5-10, no overtopping occurs in FRS#3. A separate model indicated no dam crest overtopping occurs in FRS#2 if Miller Road is washed away in the PMF (results not shown).
- The level pool routing assumes overtopping occurs on the entire dam crest, while the unsteady model only overtops part of the dam. Therefore, for the same outflow, the unsteady model gives higher overtopping depths.
- The level pool routing sets one water surface elevation throughout the FRS at any moment, which indicates full utilization of the emergency spillway discharge capacity in a dam crest overtopping scenario. Full utilization of the spillway capacity tends to underestimate the outflow on the dam crest compared to the unsteady model (for example, FRS#1 outflows).

5.9.3 Summary of Study Results and Major Conclusions

The unsteady models utilize both flow continuity equation and momentum equation to assess the hydraulic characteristics for the Buckeye Structures. Conclusions from the unsteady model analyses are summarized as following:

- The original FRS design can safely contain and convey runoff for storms up to the 500-year event without overtopping the emergency spillways.
- Overtopping of the FRS crest during a PMF occurs at FRS#1 and FRS#2 out of the three structures but is much shallower than determined in previous studies.
- Overtopping occurs for only a portion of dam crest due to crossing roadway constraint.
- Emergency spillway capacities are not fully utilized.
- In the unsteady flow model, start, magnitude, and duration of the maximum water surface elevation at each cross section is dependant on the spatial distribution of concentration points and inflow hydrographs.

- Maximum WSELs seem to be more sensitive to peak discharge than to maximum volumes; the PMP 6-hour storm event is considered as the PMF.
- The unsteady model analyses can identify problems with existing associated structures (roadways and culverts, principal spillway, etc.) that are ignored in less sophisticated models.

Warning message from the analyses indicated floodways were subject to discharge during a PMF event. Additional analyses with floodway overtopping were conducted. The results highlight the following bullets for future dam rehabilitation.

- The original analyses provide the minimum depths to be leveled for the floodways to avoid overtopped at a PMF event.
- The additional analyses indicate the maximum overtopping discharges for the floodways, which could be a weak link in the Buckeye FRS system.
- Transverse roadways affect the FRS hydraulic characteristics significantly. More discharge could be expected at the emergency spillway if the roadway did not exist.
- The floodway discharge does not affect the emergency spillway hydraulic profile.

Table 5-6

Summary of Hydraulic Results for Normal Cross Sections in the PMF from the Buckeye FRS Unsteady Models (Existing Conditions)

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS3	15750	0.01	1154.89	1170.52		0	6083.88	822.19	86.18	908.36	0.00
FRS3	15700	-0.04	1154.89	1170.52		0	6083.88	822.19	86.18	908.36	0.00
FRS3	15500	-1.38	1154.2	1170.52		0	4323.89	628.63	157.8	786.43	0.00
FRS3	15,450	Lat Struct	Emergency Spillway #3								
FRS3	15,000	-12,009.2	1,154.1	1,170.3		-3.90	4,420.2	814.6	166.37	980.97	0.18
FRS3	14,999	Lat Struct	Begin FRS#3 Dam Crest								
FRS3	14,500	-12,014.8	1,154.2	1,170.5		-3.23	5,534.2	842.2	163.20	1,005.34	0.15
FRS3	14443.5 Y1	-12,017.5	1,154.2	1,170.5		-3.23	5,523.7	842.4	163.15	1,005.58	0.15
FRS3	14295.6*	-11,914.2	1,154.1	1,170.5		-2.86	6,311.6	869.8	162.48	1,032.24	0.13
FRS3	14147.8*	-11911.8	1154.01	1170.54		-2.56	7107.67	899.17	160.24	1059.41	0.12
FRS3	14000	-11915.9	1153.89	1170.56		-2.31	7932.9	937.08	152.76	1089.85	0.1
FRS3	13500	-11914.4	1153.53	1170.59		-2.48	6982.1	856.54	140.11	1147.95	0.12
FRS3	13,000	-11,921.9	1,153.3	1,170.6		-3.15	4,929.6	614.2	135.64	749.85	0.14
FRS3	12889.70 X1	-11,922.6	1,153.3	1,170.6		-3.14	4,939.4	614.9	135.64	750.53	0.14
FRS3	12,500	-11,261.5	1,152.9	1,170.7		-2.54	6,466.3	797.0	156.29	953.26	0.11
FRS3	12,000	-11,265.4	1,152.9	1,170.8		-2.44	6,948.1	883.7	170.02	1,053.72	0.11
FRS3	11,500	-11,263.8	1,152.4	1,170.8		-2.02	8,498.1	1,015.8	155.85	1,171.68	0.09
FRS3	11,000	-11,271.9	1,152.3	1,170.8		-2.10	7,929.6	973.0	119.19	1,092.15	0.09
FRS3	10500	-11283.3	1152.34	1170.88		-2.02	8461.69	1029.2	159.75	1188.96	0.09
FRS3	10375.*	-11294.3	1152.3	1170.88		-1.95	8488.58	998.91	158.52	1157.43	0.08
FRS3	10250.*	-11305.4	1152.26	1170.89		-1.87	8559.92	952.42	158.01	1110.44	0.08
FRS3	10125.*	-11,306.6	1,152.2	1,170.9		-1.80	8,661.7	932.4	157.98	1,090.33	0.08
FRS3	10,000	-11,296.1	1,152.2	1,170.9		-1.73	8,785.5	918.3	158.43	1,076.70	0.07
FRS3	9,500	-11,323.0	1,151.9	1,170.9		-1.60	9,455.1	1,039.2	150.87	1,190.08	0.07
FRS3	9,000	-11,326.8	1,152.4	1,170.9		-1.82	8,999.0	994.7	156.04	1,150.71	0.08

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS3	8875.*	-11,311.4	1,152.4	1,170.9		-1.89	8,635.2	948.7	156.33	1,105.01	0.08
FRS3	8750.*	-11,344.3	1,152.4	1,170.9		-1.97	8,312.0	909.7	156.68	1,066.41	0.09
FRS3	8625.*	-11328.5	1152.42	1170.94		-2.06	8011.42	892.86	157.08	1049.95	0.09
FRS3	8500	-11345.9	1152.44	1170.95		-2.16	7715.45	881.29	157.53	1038.82	0.09
FRS3	8000	-11367.9	1152.44	1170.96		-2.36	6856.76	943.95	160.62	1104.57	0.11
FRS3	7875.*	-11,407.2	1,152.4	1,171.0		-2.16	7,569.4	967.1	159.78	1,126.83	0.10
FRS3	7750.*	-11,427.4	1,152.4	1,171.0		-1.97	8,289.0	991.5	159.34	1,150.83	0.09
FRS3	7625.*	-11,448.3	1,152.4	1,171.0		-1.81	9,018.7	1,019.2	159.31	1,178.53	0.08
FRS3	7,500	-11,469.8	1,152.4	1,171.0		-1.68	9,758.9	1,049.7	159.58	1,209.29	0.07
FRS3	7375.*	-11,491.6	1,152.1	1,171.0		-1.65	9,905.9	1,075.5	158.40	1,233.85	0.07
FRS3	7250.*	-11,471.1	1,151.9	1,171.0		-1.62	10,067.5	1,099.9	156.86	1,256.72	0.07
FRS3	7125.*	-11494.5	1151.59	1171.03		-1.59	10239.35	1112.03	154.81	1266.83	0.07
FRS3	7000	-11518.5	1151.31	1171.03		-1.55	10418.58	1120.87	152.21	1273.08	0.06
FRS3	6500	-11571.8	1151.88	1171.04		-1.6	9550.88	1006.15	160.32	1166.47	0.07
FRS3	6,000	-11,602.3	1,151.7	1,171.1		-1.52	9,810.5	954.5	155.71	1,110.22	0.06
FRS3	5698.20 W2	-11,605.6	1,151.7	1,171.1		-1.52	9,815.3	954.8	155.69	1,110.48	0.06
FRS3	5,500	-659.5	1,151.7	1,171.1		-0.09	9,612.7	1,013.7	166.76	1,180.46	0.00
FRS3	5,000	-663.5	1,151.6	1,171.1		-0.10	8,947.6	939.9	152.07	1,091.99	0.00
FRS3	4875.*	-664.4	1,151.8	1,171.1		-0.10	8,875.6	942.0	151.85	1,093.87	0.00
FRS3	4750.*	-677.7	1,151.9	1,171.1		-0.11	8,822.3	939.4	151.64	1,091.08	0.00
FRS3	4625.*	-653.49	1152.06	1171.12		-0.11	8793.31	929.85	151.42	1081.27	0
FRS3	4500	-666.32	1152.21	1171.12		-0.11	8787.23	922.61	152.54	1075.15	0
FRS3	4000	-669.6	1152.29	1171.12		-0.12	7737.06	888.06	167.06	1055.12	0.01
FRS3	3875.*	-670.4	1,152.1	1,171.1		-0.11	8,746.4	892.2	165.96	1,058.12	0.00
FRS3	3750.*	-662.3	1,151.8	1,171.1		-0.09	9,734.5	897.4	164.86	1,062.28	0.00
FRS3	3625.*	-671.5	1,151.6	1,171.1		-0.09	10,703.8	902.7	163.76	1,066.47	0.00
FRS3	3,500	-680.5	1,151.4	1,171.1		-0.08	11,651.1	908.5	162.66	1,071.17	0.00
FRS3	3,000	-675.6	1,151.3	1,171.1		-0.07	12,638.7	983.8	162.52	1,146.27	0.00
FRS3	2,500	-673.8	1,150.8	1,171.1		-0.08	12,055.9	956.2	157.08	1,113.30	0.00
FRS3	2000	-677.88	1150.46	1171.12		-0.08	11329.15	941.89	148.74	1090.63	0
FRS3	1875.*	-680.6	1150.43	1171.12		-0.08	11466.63	934.91	152.03	1086.94	0
FRS3	1750.*	-679.75	1150.4	1171.12		-0.08	11601.35	928.27	155.46	1083.73	0
FRS3	1625.*	-682.2	1,150.4	1,171.1		-0.08	11,734.1	923.2	159.01	1,082.21	0.00

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS3	1,500	-682.5	1,150.3	1,171.1		-0.08	11,864.1	921.2	162.66	1,083.89	0.00
FRS3	1382.00 V1	-683.4	1,150.5	1,171.1		-0.06	14,438.7	1,037.8	168.62	1,206.46	0.00
FRS3	1,000	110.6	1,150.5	1,171.1		0.01	14,439.9	1,037.9	168.61	1,206.50	0.00
FRS3	600	105.8	1,150.5	1,171.1		0.01	14,443.3	1,037.9	168.61	1,206.50	0.00
FRS3	580	Culvert	Principal Spillway #3								
FRS3	11.5	105.79	1129	1134.05		1.15	91.99	27.98	98.47	126.45	0.11
FRS3	0	105.79	1131.13	1134	1132.53	2.04	51.81	26.98	99.34	126.32	0.26
FRS2	13,699	96.3	1,121.5	1,122.7		2.52	38.2	56.2	166.37	222.52	0.54
FRS2	13599.4*	96.2	1,120.9	1,121.9		2.79	34.4	61.4	169.08	230.51	0.66
FRS2	13,499	96.0	1,120.3	1,121.1		3.08	31.2	63.3	170.07	233.32	0.77
FRS2	13399.4*	95.7	1,119.8	1,120.4	1,120.4	3.24	29.5	89.1	174.69	263.83	0.99
FRS2	13,299	92.5	1,119.4	1,119.9	1,120.0	3.72	24.9	112.5	178.16	290.66	1.39
FRS2	13199.4*	91.8	1,118.7	1,119.6		1.14	80.4	149.9	171.87	321.73	0.27
FRS2	13,099	91.1	1,118.0	1,119.6		0.58	158.0	139.7	166.59	306.31	0.10
FRS2	12999.4*	90.9	1,117.2	1,119.6		0.43	211.8	141.9	159.79	301.71	0.06
FRS2	12,899	89.8	1,116.4	1,119.6		0.33	269.3	145.1	152.94	298.03	0.04
FRS2	12799.4*	89.1	1,115.8	1,119.6		0.33	267.3	111.6	150.19	261.81	0.04
FRS2	12,699	88.6	1,115.1	1,119.6		0.38	230.5	81.0	146.22	227.23	0.04
FRS2	12599.4*	85.6	1,114.3	1,119.6		0.27	317.8	100.1	135.93	236.01	0.03
FRS2	12,499	84.5	1,113.6	1,119.6		0.18	521.1	188.1	90.45	278.53	0.02
FRS2	12399.4*	83.7	1,113.1	1,119.6		0.15	655.1	236.6	35.89	272.50	0.01
FRS2	12,299	84.6	1,112.7	1,119.6		0.12	868.6	321.4	26.20	347.56	0.01
FRS2	12199.4*	80.6	1,112.0	1,119.6		0.12	765.4	210.4	28.73	239.13	0.01
FRS2	12,099	78.2	1,111.3	1,119.6		0.12	729.9	207.3	30.62	237.89	0.01
FRS2	11999.4*	77.4	1,110.6	1,119.6		0.10	864.9	215.4	25.64	241.01	0.01
FRS2	11,899	77.5	1,109.8	1,119.6		0.09	1,018.0	236.5	20.90	257.38	0.01
FRS2	11799.4*	75.3	1,109.4	1,119.6		0.07	1,239.6	306.7	18.59	325.27	0.00
FRS2	11,699	73.5	1,108.9	1,119.6		0.06	1,542.9	382.1	20.25	402.32	0.00
FRS2	11599.4*	72.1	1,108.3	1,119.6		0.04	2,221.3	468.4	15.76	484.13	0.00
FRS2	11,499	70.4	1,107.7	1,119.6		0.03	3,027.3	555.7	11.28	566.98	0.00
FRS2	11399.4*	70.2	1,107.0	1,119.6		0.03	3,102.4	550.8	11.08	561.84	0.00
FRS2	11,299	70.2	1,106.3	1,119.6		0.03	3,274.6	684.8	10.88	695.71	0.00
FRS2	11199.4*	67.5	1,105.7	1,119.6		0.02	4,161.4	668.4	15.13	683.52	0.00

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS2	11,099	68.5	1,105.1	1,119.6		0.02	5,077.4	673.7	19.38	693.05	0.00
FRS2	10999.4*	65.5	1,104.5	1,119.6		0.02	5,275.6	652.8	16.31	669.06	0.00
FRS2	10,899	62.2	1,103.9	1,119.6		0.01	5,400.4	582.9	13.24	596.11	0.00
FRS2	10799.4*	58.3	1,103.0	1,119.6		0.01	7,226.5	809.6	16.83	826.44	0.00
FRS2	10,699	54.1	1,102.1	1,119.6		0.01	9,397.7	965.5	20.43	985.90	0.00
FRS2	10599.4* U	55.9	1,101.6	1,119.6		0.01	9,939.8	1,045.2	15.54	1,060.70	0.00
FRS2	10,499	2,786.9	1,101.1	1,119.5		0.32	10,598.9	1,124.7	10.65	1,135.30	0.01
FRS2	10,299	2,783.8	1,101.0	1,119.5		0.49	9,201.7	1,145.2	17.30	1,162.49	0.02
FRS2	10,299	Lat Struct	Begin FRS#2 Dam Crest								
FRS2	10,099	2,583.9	1,101.2	1,119.5		0.44	9,102.8	1,093.2	17.72	1,110.94	0.02
FRS2	9,899	2,347.1	1,101.0	1,119.5		0.36	10,188.9	1,200.6	22.03	1,222.58	0.02
FRS2	9,699	2,063.2	1,100.5	1,119.5		0.30	10,744.7	1,147.6	19.22	1,166.85	0.01
FRS2	9,499	1,770.7	1,100.0	1,119.5		0.26	10,369.4	948.6	15.70	964.26	0.01
FRS2	9,299	1,400.7	1,099.5	1,119.5		0.18	12,057.7	1,134.2	12.42	1,146.61	0.01
FRS2	9,099	976.4	1,099.5	1,119.5		0.12	12,212.2	1,140.4	8.73	1,149.17	0.00
FRS2	8,899	589.8	1,099.5	1,119.5		0.08	12,107.0	1,308.0	16.30	1,324.25	0.00
FRS2	8,699	153.2	1,099.3	1,119.5		0.02	11,804.2	1,218.4	13.45	1,231.87	0.00
FRS2	8599.41* T	-74.9	1,099.3	1,119.5		-0.01	12,408.1	1,348.6	16.16	1,364.72	0.00
FRS2	8,499	8,020.0	1,099.3	1,119.5		1.03	13,157.9	1,381.1	18.86	1,399.96	0.04
FRS2	8,299	7,583.6	1,099.2	1,119.5		0.98	12,136.2	1,348.2	9.24	1,357.40	0.04
FRS2	8,099	7,166.2	1,099.2	1,119.5		1.11	10,300.8	1,353.3	15.66	1,368.98	0.05
FRS2	7,899	6,729.4	1,099.0	1,119.5		1.08	9,341.2	1,110.8	15.53	1,126.32	0.04
FRS2	7,699	6,304.6	1,099.0	1,119.5		1.03	9,214.2	1,050.3	17.71	1,067.99	0.04
FRS2	7,499	5,875.3	1,098.9	1,119.5		1.07	9,128.9	1,171.9	17.64	1,189.56	0.04
FRS2	7,299	5,509.5	1,099.3	1,119.5		0.91	10,262.3	1,260.8	18.20	1,278.97	0.04
FRS2	7,099	5,331.0	1,099.2	1,119.5		0.80	11,486.7	1,244.3	16.74	1,261.00	0.03
FRS2	6,899	4,986.1	1,099.1	1,119.5		0.71	11,890.0	1,272.3	5.34	1,277.61	0.03
FRS2	6,699	4,588.6	1,098.9	1,119.5		0.67	11,568.3	1,220.4	8.24	1,228.68	0.03
FRS2	6,499	4,402.6	1,098.8	1,119.5		0.67	10,956.7	1,137.6	9.76	1,147.38	0.03
FRS2	6,299	4,083.8	1,099.0	1,119.5		0.53	13,085.4	1,347.5	6.88	1,354.33	0.02
FRS2	6,099	3,737.0	1,098.9	1,119.5		0.59	10,480.8	1,200.4	8.30	1,216.80	0.02
FRS2	5,899	3,380.4	1,098.9	1,119.5		0.57	9,750.5	1,126.8	16.72	1,143.54	0.02
FRS2	5,699	2,992.4	1,098.9	1,119.5		0.52	9,818.1	1,301.1	8.32	1,309.41	0.02

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS2	5,499	2,587.4	1,098.9	1,119.5		0.36	10,955.8	1,071.8	17.48	1,089.32	0.01
FRS2	5,299	2,160.0	1,099.2	1,119.5		0.34	9,231.8	1,007.9	12.01	1,019.88	0.01
FRS2	5,099	1,747.7	1,098.9	1,119.5		0.29	8,684.3	986.7	9.42	996.08	0.01
FRS2	4,899	1,359.0	1,098.9	1,119.5		0.23	8,173.9	870.5	14.11	884.57	0.01
FRS2	4,699	941.4	1,099.1	1,119.5		0.17	9,038.4	974.3	0.00	974.32	0.01
FRS2	4,286	313.9	1,099.7	1,119.5		0.07	7,419.4	977.2	49.81	1,027.00	0.00
FRS2	4,279	Culvert	underneath Miller Road								
FRS2	4,166	-300.1	1,099.5	1,117.0		-0.08	5,256.9	726.1	58.78	784.90	0.00
FRS2	4,099	Lat Struct	Continue FRS#2 Dam Crest after Miller Road								
FRS2	3,899	-305.5	1,098.7	1,117.0		-0.09	5,082.6	740.1	18.40	758.45	0.00
FRS2	3,699	-306.2	1,098.2	1,117.0		-0.07	6,140.0	751.3	18.78	770.07	0.00
FRS2	3,499	-306.9	1,098.6	1,117.0		-0.07	5,985.9	730.8	14.93	745.78	0.00
FRS2	3,299	-306.7	1,098.3	1,117.0		-0.07	6,340.2	810.7	13.68	824.40	0.00
FRS2	3,099	-300.6	1,098.7	1,117.0		-0.07	6,429.2	799.9	8.51	808.37	0.00
FRS2	2,899	-308.9	1,098.3	1,116.9		-0.07	7,016.3	907.5	7.10	914.55	0.00
FRS2	2,699	-305.8	1,097.9	1,116.9		-0.06	7,486.6	942.0	10.58	952.62	0.00
FRS2	2,649	Lat Struct	Emergency Spillway #2								
FRS2	2624.70*	-1,576.1	1,097.7	1,116.9		-0.31	7,757.8	1,030.4	10.23	1,040.64	0.01
FRS2	2549.99* S	-3,961.4	1,097.5	1,116.9		-0.71	8,083.3	1,066.4	10.70	1,077.09	0.03
FRS2	2,500	1,730.0	1,097.4	1,116.9		0.29	8,335.4	1,079.4	12.53	1,091.90	0.01
FRS2	2,389	152.1	1,096.5	1,116.9		0.04	6,777.8	919.8	27.03	946.80	0.00
FRS2	2,220	Culvert	Principal Spillway #2								
FRS2	150	152.1	1,085.0	1,089.0		2.27	67.0	30.0	167.25	197.23	0.27
FRS2	0	152.0	1,084.9	1,088.9	1,086.8	1.23	124.2	92.5	50.91	143.44	0.17
FRS1	48,900	146.7	1,086.1	1,096.1		0.08	2,605.2	453.7	8.32	462.05	0.00
FRS1	48,700	145.9	1,086.0	1,096.1		0.04	6,711.3	1,082.8	25.34	1,108.15	0.00
FRS1	48399.9* R	145.5	1,085.4	1,096.1		0.03	7,253.0	1,084.8	16.28	1,101.03	0.00
FRS1	48,300	186.8	1,085.2	1,096.1		0.03	7,506.8	1,085.4	13.26	1,098.66	0.00
FRS1	47,900	186.2	1,084.3	1,096.1		0.04	5,970.8	1,189.9	18.77	1,208.65	0.00
FRS1	47699.9* Q	184.8	1,084.2	1,096.1		0.04	6,325.0	911.5	17.62	929.15	0.00
FRS1	47,500	3,873.3	1,084.0	1,096.0		0.92	5,212.7	575.0	16.47	591.47	0.05
FRS1	47,300	3,872.3	1,083.8	1,096.0		0.84	5,086.1	510.9	20.45	531.36	0.04
FRS1	46,300	3,853.1	1,084.0	1,095.4		6.32	679.9	87.9	18.62	106.53	0.35

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS1	46,100	3,842.4	1,083.9	1,095.2		6.18	648.2	96.3	1.84	98.10	0.38
FRS1	45,900	3,846.3	1,084.0	1,095.2		5.26	755.8	108.5	2.68	111.14	0.32
FRS1	45,700	3,846.1	1,084.1	1,095.1		4.73	851.6	116.5	0.00	116.54	0.28
FRS1	45,500	3,839.9	1,084.0	1,095.0		4.92	806.8	107.4	11.75	119.19	0.30
FRS1	45,300	3,841.0	1,082.9	1,095.0		4.18	984.9	130.7	0.00	130.67	0.24
FRS1	45,100	3,840.7	1,082.7	1,095.1		3.32	1,377.7	185.7	9.59	195.26	0.18
FRS1	44,900	3,842.7	1,082.4	1,095.1		1.84	2,599.1	328.4	10.76	339.15	0.10
FRS1	44,100	3,839.7	1,082.5	1,094.7		4.76	852.7	93.1	11.72	104.79	0.26
FRS1	43,900	3,838.8	1,083.0	1,094.5		5.42	752.3	100.8	0.00	100.82	0.32
FRS1	43,700	3,837.3	1,083.0	1,094.4		5.34	777.5	91.3	12.33	103.65	0.29
FRS1	43,500	3,837.7	1,083.0	1,094.3		5.13	784.8	100.1	6.28	106.36	0.30
FRS1	43,300	3,837.2	1,082.7	1,094.2		5.26	766.6	93.9	7.49	101.41	0.30
FRS1	43,100	3,836.3	1,083.0	1,094.1		4.67	855.4	111.8	8.46	120.25	0.28
FRS1	42,900	3,836.5	1,082.7	1,094.3		2.51	2,059.4	283.4	0.00	283.43	0.14
FRS1	41,700	3,834.7	1,082.7	1,094.1		2.62	1,798.3	244.3	11.68	256.00	0.14
FRS1	41,500	3,835.2	1,082.9	1,094.0		3.39	1,337.3	208.9	13.86	222.78	0.19
FRS1	41,300	3,831.5	1,083.2	1,093.8		4.70	863.6	127.5	3.34	130.85	0.29
FRS1	41,100	3,821.1	1,083.2	1,093.4		6.08	666.8	105.3	1.69	106.98	0.38
FRS1	40,900	3,791.6	1,082.8	1,093.2		6.07	656.1	105.2	0.00	105.15	0.39
FRS1	40,700	3,762.3	1,083.0	1,093.1		5.95	689.4	95.1	13.69	108.79	0.35
FRS1	40,500	3,676.8	1,082.9	1,092.9		5.94	655.5	92.3	14.18	106.51	0.36
FRS1	40,300	3,701.8	1,082.4	1,092.9		4.71	824.5	115.0	7.79	122.82	0.28
FRS1	40,100	3,724.6	1,082.4	1,092.9		3.98	1,050.6	166.0	6.96	172.95	0.24
FRS1	39,900	3,731.5	1,082.2	1,092.9		3.27	1,450.7	238.5	10.22	248.74	0.19
FRS1	39,700	3,678.3	1,082.2	1,092.8		3.62	1,201.6	209.3	13.78	223.04	0.22
FRS1	39,500	3,700.1	1,082.1	1,092.8		3.22	1,533.8	265.5	0.00	265.49	0.19
FRS1	39,300	3,708.2	1,081.2	1,092.9		2.06	2,071.2	297.0	0.00	297.01	0.12
FRS1	39,100	3,712.3	1,081.6	1,092.8		2.08	2,143.2	301.9	0.00	301.87	0.12
FRS1	38,900	3,666.0	1,081.4	1,092.8		2.63	1,675.0	252.4	2.29	254.69	0.15
FRS1	38,700	3,670.7	1,081.3	1,092.8		2.45	1,895.9	284.6	0.00	284.57	0.14
FRS1	38,500	3,660.1	1,081.3	1,092.7		2.51	1,898.7	287.0	0.00	287.01	0.14
FRS1	38,300	3,607.9	1,081.2	1,092.7		2.71	1,699.6	268.4	3.79	272.20	0.15
FRS1	38,100	3,595.1	1,081.6	1,092.7		2.65	1,718.6	242.3	5.83	248.08	0.15

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS1	37,900	2,783.2	1,081.7	1,092.6		2.69	1,158.3	176.6	0.00	176.59	0.16
FRS1	37,700	2,778.4	1,081.5	1,092.5		2.71	1,045.1	172.2	12.25	184.40	0.19
FRS1	37,600	Lat Struct	Begin FRS#1 Dam Crest								
FRS1	37,500	2,651.9	1,081.4	1,092.5		2.46	1,125.9	191.6	4.61	196.21	0.17
FRS1	37,300	2,378.9	1,081.5	1,092.5		2.23	1,162.6	213.5	0.00	213.47	0.15
FRS1	37,100	2,069.2	1,081.5	1,092.5		1.69	1,381.8	219.1	12.17	231.30	0.10
FRS1	36,900	1,720.6	1,081.4	1,092.5		1.35	1,367.7	239.1	6.58	245.64	0.09
FRS1	36699.98 P	1,338.7	1,081.7	1,092.6		0.71	2,230.2	381.9	0.00	381.94	0.04
FRS1	36,500	2,544.9	1,081.3	1,092.5		1.69	2,375.6	585.7	11.40	597.11	0.10
FRS1	36,300	2,138.0	1,081.2	1,092.5		1.27	2,776.6	590.5	6.63	597.13	0.07
FRS1	36,100	1,717.0	1,081.1	1,092.5		0.95	2,989.5	669.9	0.00	669.85	0.05
FRS1	35,900	1,288.4	1,080.5	1,092.5		0.72	3,086.5	687.5	12.79	700.29	0.04
FRS1	35699.9* O	858.5	1,080.6	1,092.5		0.44	3,337.8	734.2	6.39	740.62	0.02
FRS1	35,500	2,922.4	1,080.7	1,092.5		1.36	3,562.6	781.0	0.00	780.96	0.08
FRS1	35,300	2,520.0	1,080.2	1,092.5		1.11	4,175.5	938.0	10.28	948.31	0.06
FRS1	35,100	2,150.9	1,080.5	1,092.5		0.81	4,868.6	1,000.1	8.01	1,008.10	0.04
FRS1	34,900	1,784.0	1,079.9	1,092.5		0.61	5,456.4	1,098.5	0.00	1,098.52	0.03
FRS1	34,700	1,445.5	1,078.5	1,092.5		0.42	6,479.2	1,178.2	3.66	1,181.89	0.02
FRS1	34,500	1,146.0	1,078.3	1,092.5		0.26	7,313.2	1,340.3	0.27	1,340.52	0.01
FRS1	34,300	856.0	1,078.5	1,092.5		0.17	8,893.8	1,406.3	0.00	1,427.17	0.01
FRS1	34099.98 N	594.7	1,077.9	1,092.5		0.11	8,356.3	1,325.8	0.00	1,325.80	0.01
FRS1	33,900	4,945.4	1,077.2	1,092.5		1.40	6,328.2	1,232.9	10.19	1,243.08	0.07
FRS1	33,700	4,676.9	1,077.0	1,092.5		1.11	6,908.0	1,190.1	10.64	1,200.75	0.05
FRS1	33,500	4,437.3	1,076.7	1,092.5		1.07	7,689.0	1,331.4	7.05	1,338.41	0.05
FRS1	33,300	4,182.1	1,076.4	1,092.5		0.93	8,460.8	1,362.6	3.03	1,365.64	0.04
FRS1	33,100	3,910.6	1,076.1	1,092.5		0.78	9,110.3	1,418.6	0.00	1,418.59	0.04
FRS1	32,900	3,623.8	1,075.6	1,092.5		0.68	10,011.3	1,450.7	13.81	1,464.53	0.03
FRS1	32,700	3,286.9	1,075.2	1,092.5		0.61	10,041.3	1,459.9	9.76	1,469.68	0.03
FRS1	32,500	2,927.3	1,074.8	1,092.5		0.39	12,064.5	1,523.9	7.59	1,531.44	0.02
FRS1	32,300	2,474.7	1,074.4	1,092.5		0.38	11,612.4	1,560.1	0.00	1,560.09	0.02
FRS1	32,100	1,987.2	1,074.2	1,092.5		0.16	14,627.9	1,567.6	10.84	1,578.41	0.01
FRS1	31,900	1,440.3	1,074.0	1,092.5		0.11	15,221.4	1,526.5	0.00	1,526.46	0.00
FRS1	31,700	818.5	1,073.7	1,092.5		0.06	15,419.5	1,488.1	0.00	1,488.10	0.00

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS1	31,500	178.7	1,073.4	1,092.5		0.01	16,063.6	1,547.3	0.00	1,547.27	0.00
FRS1	31,300	-492.6	1,073.3	1,092.5		-0.04	16,221.1	1,573.8	2.13	1,575.93	0.00
FRS1	31,100	-1,195.6	1,072.9	1,092.5		-0.10	15,680.3	1,548.2	6.68	1,554.92	0.00
FRS1	30,900	-1,922.2	1,072.6	1,092.5		-0.15	16,193.9	1,750.8	0.00	1,750.77	0.01
FRS1	30,700	-2,683.3	1,072.4	1,092.5		-0.28	14,719.8	1,743.9	0.39	1,744.32	0.01
FRS1	30,500	-3,472.0	1,072.2	1,092.5		-0.34	17,194.9	1,810.7	5.13	1,815.83	0.01
FRS1	30,300	-4,245.6	1,071.9	1,092.5		-0.42	16,927.7	1,909.6	0.00	1,909.56	0.02
FRS1	30,100	-4,945.1	1,071.7	1,092.5		-0.44	17,419.2	1,845.1	1.84	1,846.98	0.02
FRS1	29,900	-5,568.6	1,071.6	1,092.5		-0.36	20,191.2	1,809.7	8.91	1,818.56	0.01
FRS1	29,700	-6,103.4	1,071.3	1,092.5		-0.39	20,479.4	1,783.8	0.00	1,783.83	0.02
FRS1	29,500	-6,580.5	1,071.4	1,092.5		-0.42	20,192.0	1,843.5	2.57	1,846.05	0.02
FRS1	29,300	-7,001.3	1,070.8	1,092.5		-0.61	18,548.6	1,885.4	8.77	1,894.20	0.02
FRS1	29,100	-7,403.4	1,070.5	1,092.5		-0.67	18,786.3	1,940.1	0.00	1,940.06	0.03
FRS1	28,900	-7,789.9	1,069.9	1,092.5		-0.71	18,714.7	1,957.2	2.90	1,960.13	0.03
FRS1	28,700	-8,174.0	1,069.9	1,092.5		-0.71	19,468.0	1,940.3	6.95	1,947.23	0.03
FRS1	28,500	-8,583.4	1,069.8	1,092.5		-0.72	19,885.8	2,030.4	5.65	2,036.09	0.03
FRS1	28,300	-9,000.1	1,069.5	1,092.5		-0.65	20,757.5	2,056.2	0.00	2,056.20	0.03
FRS1	28,100	-9,394.8	1,069.1	1,092.5		-0.63	22,417.4	2,102.9	5.17	2,108.02	0.02
FRS1	27,900	-9,815.7	1,068.7	1,092.5		-0.62	23,985.8	2,135.4	0.00	2,135.44	0.02
FRS1	27,700	-10,250.3	1,068.9	1,092.5		-0.62	25,230.1	2,201.1	0.00	2,201.05	0.02
FRS1	27,500	-10,676.8	1,069.0	1,092.5		-0.74	24,122.6	2,103.1	5.48	2,108.59	0.03
FRS1	27,300	-11,087.0	1,068.7	1,092.5		-0.78	24,593.6	2,148.8	13.95	2,162.72	0.03
FRS1	27,100	-11,543.8	1,068.6	1,092.5		-0.76	25,716.9	2,233.2	0.16	2,233.36	0.03
FRS1	26,900	-11,986.1	1,068.6	1,092.5		-0.68	26,884.1	2,257.2	7.75	2,264.97	0.03
FRS1	26,700	-12,324.6	1,068.7	1,092.5		-0.49	30,835.0	2,270.6	4.23	2,274.81	0.02
FRS1	26,500	-12,688.3	1,068.7	1,092.5		-0.50	30,528.9	2,288.8	11.50	2,300.32	0.02
FRS1	26,300	-12,860.6	1,068.2	1,092.5		-0.52	29,701.6	2,243.0	0.00	2,320.52	0.02
FRS1	26,100	-13,119.4	1,068.1	1,092.5		-0.50	32,647.3	2,379.8	4.10	2,383.87	0.02
FRS1	25,900	-13,363.2	1,067.9	1,092.5		-0.49	33,664.2	2,406.8	0.33	2,407.14	0.02
FRS1	25,700	-13,548.9	1,067.7	1,092.5		-0.47	34,923.0	2,487.9	5.00	2,492.89	0.02
FRS1	25,500	-13,695.8	1,067.4	1,092.5		-0.46	36,339.1	2,478.6	12.21	2,490.79	0.02
FRS1	25,300	-13,983.5	1,067.3	1,092.5		-0.47	36,418.3	2,492.9	0.00	2,492.86	0.02
FRS1	25099.98 M	-14,048.2	1,067.5	1,092.5		-0.70	30,970.8	2,567.1	2.92	2,569.98	0.02

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS1	24,935	5,122.7	1,067.5	1,092.5		0.24	30,715.3	2,655.2	1.81	2,657.04	0.01
FRS1	24,885	Culvert	Sun Valley Parkway								
FRS1	24,700	4,816.7	1,066.9	1,089.3		0.36	20,649.7	2,257.9	18.19	2,276.09	0.02
FRS1	24,699	Lat Struct	Continue FRS#1 Dam Crest after Sun Valley Parkway								
FRS1	24,500	4,726.6	1,066.6	1,089.4		0.27	25,833.0	2,254.1	6.34	2,260.45	0.01
FRS1	24,300	4,647.7	1,066.0	1,089.4		0.24	27,489.4	2,220.5	8.50	2,228.98	0.01
FRS1	24,100	4,561.6	1,065.7	1,089.4		0.17	31,515.5	2,194.7	11.82	2,206.56	0.01
FRS1	23,900	4,498.7	1,065.7	1,089.4		0.17	31,151.2	2,281.1	6.70	2,287.76	0.01
FRS1	23,700	4,385.5	1,065.5	1,089.4		0.16	31,308.3	2,353.6	11.99	2,365.59	0.01
FRS1	23,500	4,327.3	1,065.4	1,089.4		0.16	31,108.2	2,311.9	13.35	2,325.28	0.01
FRS1	23,300	4,268.6	1,065.4	1,089.4		0.16	30,923.2	2,328.6	7.99	2,336.62	0.01
FRS1	23,100	4,211.8	1,065.3	1,089.4		0.16	30,941.0	2,304.8	8.89	2,313.71	0.01
FRS1	22,900	4,202.4	1,065.2	1,089.4		0.16	30,716.1	2,321.1	12.51	2,333.65	0.01
FRS1	22,700	4,147.0	1,065.0	1,089.4		0.16	30,672.3	2,358.2	10.74	2,368.97	0.01
FRS1	22499.99 Z	4,042.0	1,065.2	1,089.4		0.16	30,880.5	2,398.4	0.00	2,398.42	0.01
FRS1	22,300	3,983.7	1,065.1	1,089.4		0.23	26,761.3	2,384.3	7.64	2,391.93	0.01
FRS1	22,100	3,929.7	1,065.0	1,089.4		0.23	26,818.0	2,452.8	11.86	2,464.63	0.01
FRS1	21,900	3,877.0	1,064.7	1,089.4		0.23	26,719.9	2,368.3	9.40	2,377.65	0.01
FRS1	21,700	3,826.3	1,065.0	1,089.4		0.22	26,969.5	2,408.3	6.24	2,414.55	0.01
FRS1	21,500	3,773.1	1,065.0	1,089.4		0.21	27,373.8	2,449.1	11.99	2,461.06	0.01
FRS1	21,300	3,671.4	1,064.9	1,089.4		0.19	29,066.2	2,497.6	9.35	2,506.90	0.01
FRS1	21,100	3,617.1	1,064.8	1,089.4		0.18	30,122.8	2,535.0	7.70	2,542.69	0.01
FRS1	20,900	3,609.0	1,064.5	1,089.4		0.18	30,281.3	2,545.2	10.87	2,556.02	0.01
FRS1	20,700	3,512.1	1,064.1	1,089.4		0.18	29,695.6	2,565.2	9.58	2,574.77	0.01
FRS1	20,500	3,458.8	1,064.2	1,089.4		0.17	30,181.1	2,567.3	7.32	2,574.62	0.01
FRS1	20,300	3,406.2	1,064.1	1,089.4		0.16	31,203.9	2,498.2	11.56	2,509.71	0.01
FRS1	20,100	3,355.0	1,063.8	1,089.4		0.16	31,261.7	2,468.4	0.00	2,468.38	0.01
FRS1	19,900	3,250.3	1,064.1	1,089.4		0.15	31,243.6	2,505.2	0.00	2,505.23	0.01
FRS1	18,700	2,968.1	1,064.1	1,089.4		0.16	28,743.4	2,508.0	11.14	2,519.13	0.01
FRS1	18,500	2,911.0	1,063.8	1,089.4		0.17	28,215.5	2,499.3	12.23	2,511.54	0.01
FRS1	18,300	2,802.4	1,063.7	1,089.4		0.16	29,019.6	2,541.6	9.40	2,551.00	0.01
FRS1	18,100	2,685.4	1,063.7	1,089.4		0.16	29,023.8	2,542.9	10.30	2,553.15	0.01
FRS1	17,900	2,622.1	1,063.6	1,089.4		0.15	29,377.5	2,572.0	10.27	2,582.29	0.01

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS1	17,700	2,556.5	1,063.7	1,089.4		0.15	29,473.9	2,775.4	7.69	2,783.12	0.01
FRS1	17,500	2,423.8	1,063.5	1,089.4		0.13	32,955.3	2,748.8	6.09	2,754.91	0.00
FRS1	17,300	2,278.9	1,063.3	1,089.4		0.12	32,741.2	2,786.3	5.31	2,809.53	0.00
FRS1	17,100	2,039.5	1,063.6	1,089.4		0.10	35,677.6	3,085.4	8.16	3,093.53	0.00
FRS1	16,900	2,035.2	1,063.0	1,089.4		0.09	37,002.9	3,144.1	7.07	3,151.20	0.00
FRS1	16,700	1,952.0	1,063.1	1,089.4		0.08	40,945.5	3,209.7	7.33	3,216.99	0.00
FRS1	16,500	1,868.8	1,063.0	1,089.4		0.06	46,682.7	3,246.6	10.76	3,257.34	0.00
FRS1	16,300	1,787.1	1,062.3	1,089.4		0.05	45,648.8	3,121.7	6.34	3,128.01	0.00
FRS1	16,100	1,704.9	1,062.6	1,089.4		0.05	46,654.8	3,264.8	10.27	3,275.06	0.00
FRS1	15,900	1,620.4	1,062.4	1,089.4		0.04	54,524.4	3,677.3	10.81	3,688.11	0.00
FRS1	15,700	1,537.3	1,062.5	1,089.4		0.04	49,811.6	3,464.7	9.06	3,473.73	0.00
FRS1	15,500	1,530.0	1,062.7	1,089.4		0.04	50,176.3	3,472.8	5.24	3,478.08	0.00
FRS1	15,100	1,449.2	1,062.5	1,089.4		0.04	48,796.6	3,507.3	11.92	3,519.26	0.00
FRS1	14,900	1,441.7	1,062.5	1,089.4		0.05	47,505.9	3,534.6	0.00	3,534.57	0.00
FRS1	14,700	1,432.5	1,062.5	1,089.4		0.04	48,597.8	3,269.1	4.82	3,273.93	0.00
FRS1	14,500	1,384.9	1,062.5	1,089.4		0.03	50,466.0	3,274.7	6.97	3,281.63	0.00
FRS1	14,300	1,410.7	1,062.5	1,089.4		0.03	50,976.7	3,333.1	0.00	3,333.09	0.00
FRS1	14,100	1,401.3	1,062.7	1,089.4		0.03	49,499.4	3,283.2	11.13	3,566.01	0.00
FRS1	13899.99 L	1,390.9	1,062.5	1,089.4		0.04	49,372.9	3,449.5	11.06	3,460.52	0.00
FRS1	13,700	1,700.9	1,062.8	1,089.4		0.05	50,631.4	3,923.1	11.40	3,934.50	0.00
FRS1	13,650	Culvert	Johnson Road								
FRS1	13,500	-1,572.0	1,062.9	1,087.2		-0.05	41,182.0	3,276.7	20.37	3,297.03	0.00
FRS1	13,499	Lat Struct	Continue FRS#1 Dam Crest after Johnson Road								
FRS1	13,300	-1,574.0	1,062.6	1,087.2		-0.05	42,474.8	3,447.3	19.23	3,466.51	0.00
FRS1	12,900	-1,579.9	1,062.8	1,087.2		-0.05	45,245.5	3,739.2	15.45	3,754.62	0.00
FRS1	12,700	-1,578.2	1,062.7	1,087.2		-0.05	47,244.7	3,530.1	16.55	3,546.66	0.00
FRS1	12,500	-1,586.0	1,062.6	1,087.2		-0.04	50,221.6	3,693.8	12.43	3,706.23	0.00
FRS1	12,300	-1,591.4	1,062.8	1,087.2		-0.05	46,319.1	3,781.9	16.64	3,798.57	0.00
FRS1	11,700	-1,600.2	1,062.9	1,087.2		-0.09	31,667.6	3,063.8	14.07	3,077.85	0.00
FRS1	11,500	-1,588.9	1,062.8	1,087.2		-0.09	30,176.7	2,995.5	16.97	3,012.43	0.00
FRS1	11,300	-1,576.8	1,062.5	1,087.2		-0.08	30,850.1	3,034.9	15.40	3,050.29	0.00
FRS1	11,100	-1,547.3	1,062.6	1,087.2		-0.08	33,209.1	2,938.4	14.38	2,952.74	0.00
FRS1	10,900	-1,566.0	1,062.6	1,087.2		-0.09	30,373.2	2,936.2	11.86	3,066.90	0.00

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS1	10,700	-1,586.2	1,062.6	1,087.2		-0.09	31,630.7	3,153.4	14.47	3,167.82	0.00
FRS1	10,500	-1,606.9	1,062.6	1,087.2		-0.07	34,386.4	3,012.4	19.65	3,032.04	0.00
FRS1	10,300	-1,627.7	1,062.7	1,087.2		-0.06	36,862.8	3,059.7	21.34	3,080.99	0.00
FRS1	10,100	-1,591.0	1,062.8	1,087.2		-0.06	40,916.9	3,249.2	16.52	3,265.68	0.00
FRS1	9,900	-1,615.6	1,063.0	1,087.2		-0.06	39,800.6	3,095.7	17.14	3,112.82	0.00
FRS1	9699.999 E	-1,593.5	1,063.3	1,087.2		-0.06	36,608.7	2,784.1	15.64	2,799.71	0.00
FRS1	9,500	20,222.4	1,063.5	1,087.2		0.82	38,610.2	3,379.6	14.05	3,393.66	0.03
FRS1	9,300	20,194.2	1,063.4	1,087.2		0.65	40,985.6	3,350.6	18.21	3,368.82	0.03
FRS1	9,100	20,242.3	1,063.3	1,087.2		0.78	38,846.1	3,063.7	17.79	3,081.48	0.03
FRS1	8,900	20,264.5	1,063.1	1,087.2		0.77	40,828.2	3,014.9	17.84	3,073.63	0.03
FRS1	8,700	20,259.4	1,063.3	1,087.1		0.87	38,759.3	3,889.3	18.81	3,908.08	0.03
FRS1	8,500	20,229.0	1,062.6	1,087.1		1.37	30,646.5	4,102.4	22.12	4,124.51	0.05
FRS1	8,300	20,202.5	1,062.7	1,087.1		1.45	25,843.1	4,394.5	18.71	4,413.17	0.05
FRS1	8,100	20,195.2	1,062.8	1,087.1		1.20	30,511.3	3,363.9	16.06	3,379.94	0.04
FRS1	7,900	20,172.7	1,062.5	1,087.1		0.90	35,226.2	3,589.7	16.42	3,606.11	0.04
FRS1	7,700	20,154.1	1,062.7	1,087.1		1.56	26,755.8	3,591.8	16.22	3,607.99	0.06
FRS1	7,500	20,135.6	1,062.9	1,087.1		1.92	17,252.1	2,148.1	18.55	2,166.68	0.08
FRS1	7,300	20,090.7	1,063.0	1,087.1		2.14	16,300.6	2,050.0	19.60	2,069.60	0.09
FRS1	7,100	20,102.0	1,062.9	1,087.1		3.06	14,879.7	2,589.1	18.38	2,607.47	0.13
FRS1	6,900	20,072.5	1,062.7	1,087.0		3.57	12,810.4	2,469.6	16.82	2,486.40	0.15
FRS1	6,700	19,988.1	1,062.9	1,087.0		4.77	9,416.9	2,449.3	22.81	2,472.11	0.21
FRS1	6,500	19,953.9	1,062.9	1,086.9		4.82	8,192.0	1,879.9	19.05	1,898.96	0.22
FRS1	6,300	19,879.8	1,062.6	1,086.8		5.34	7,110.5	1,568.9	19.06	1,587.92	0.23
FRS1	6,100	19,759.0	1,062.3	1,086.6		6.14	5,653.7	1,332.3	21.25	1,353.51	0.27
FRS1	5,900	19,662.7	1,062.7	1,086.5		6.48	4,723.4	1,051.1	19.61	1,070.75	0.29
FRS1	5,700	19,468.7	1,063.2	1,086.3		6.87	4,194.7	913.5	22.60	936.10	0.31
FRS1	5,500	19,361.1	1,062.9	1,086.2		7.07	4,094.6	836.1	23.45	859.58	0.31
FRS1	5,300	19,359.6	1,062.7	1,086.2		6.58	4,401.8	907.0	18.39	925.42	0.30
FRS1	5,100	19,348.2	1,062.7	1,086.1		6.17	5,361.0	1,377.8	20.88	1,398.64	0.27
FRS1	4,900	19,286.3	1,062.7	1,086.1		6.02	5,969.9	1,400.6	18.17	1,418.75	0.27
FRS1	4,700	19,333.8	1,062.4	1,086.1		5.09	7,513.7	1,474.9	6.63	1,481.53	0.23
FRS1	4,500	19,350.8	1,063.2	1,086.2		2.50	10,365.9	1,179.2	9.43	1,188.61	0.10
FRS1	4,300	19,358.2	1,062.7	1,086.2		2.72	11,811.3	1,676.8	9.38	1,686.13	0.11

Reach	River Sta	Q Total (cfs)	Min Ch El (ft)	WSEL (ft)	Crit W.S. (ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Sta W.S. Lft (ft)	Sta W.S. Rgt (ft)	Froude No.
FRS1	4,100	19,390.0	1,062.4	1,086.2		2.09	16,994.0	1,754.2	6.17	1,760.35	0.08
FRS1	3,900	19,381.0	1,062.7	1,086.2		1.02	28,035.4	1,769.2	3.10	1,772.31	0.04
FRS1	3299.99* G	19,388.9	1,062.8	1,086.2		1.22	28,666.6	2,504.7	4.42	2,509.07	0.05
FRS1	2,700	19,371.3	1,063.0	1,086.2		1.30	30,097.9	3,028.0	5.76	3,033.77	0.05
FRS1	2,699	Lat Struct									
FRS1	1,900	628.2	1,062.6	1,086.2		0.07	20,886.5	3,031.2	18.43	3,049.64	0.00
FRS1	1,700	474.6	1,062.6	1,086.2		0.05	20,886.5	3,031.2	18.43	3,049.64	0.00
FRS1	1,680	Culvert									
FRS1	500	474.5	1,028.1	1,031.3		4.43	110.8	44.1	10.44	54.53	0.47
FRS1	300	474.5	1,028.1	1,031.1		3.78	140.0	56.8	0.00	56.80	0.39
FRS1	100	474.5	1,028.1	1,030.9		2.92	162.4	72.6	26.92	99.47	0.34
FRS1	0	474.4	1,028.0	1,030.8	1,029.4	2.73	175.2	77.5	0.00	77.51	0.32

Table 5-7

Summary of Hydraulic Results for Principal/Emergency Spillway and Dam Crest in the PMF

Principal Spillway #3

Q Culv Group (cfs)	105.79	Culv Full Len (ft)	558.5
# Barrels	1	Culv Vel US (ft/s)	21.55
Q Barrel (cfs)	105.79	Culv Vel DS (ft/s)	21.55
E.G. US. (ft)	1171.15	Culv Inv El Up (ft)	1151.03
W.S. US. (ft)	1171.12	Culv Inv El Dn (ft)	1131.13
E.G. DS (ft)	1134.07	Culv Frctn Ls (ft)	26.28
W.S. DS (ft)	1134.05	Culv Exit Loss (ft)	7.19
Delta EG (ft)	37.08	Culv Entr Loss (ft)	3.61
Delta WS (ft)	37.07	Q Weir (cfs)	
E.G. IC (ft)	1171.15	Weir Sta Lft (ft)	
E.G. OC (ft)	1176.53	Weir Sta Rgt (ft)	
Culvert Control	Inlet	Weir Submerg	
Culv WS Inlet (ft)	1153.53	Weir Max Depth (ft)	
Culv WS Outlet (ft)	1133.63	Weir Avg Depth (ft)	
Culv Nml Depth (ft)	2.5	Weir Flow Area (sq ft)	
Culv Crt Depth (ft)	2.5	Min El Weir Flow (ft)	1171.45

Principal Spillway #2

Q Culv Group (cfs)	152.11	Culv Full Len (ft)	2200
# Barrels	1	Culv Vel US (ft/s)	12.1
Q Barrel (cfs)	152.11	Culv Vel DS (ft/s)	12.12
E.G. US. (ft)	1117.09	Culv Inv El Up (ft)	1096.53
W.S. US. (ft)	1116.94	Culv Inv El Dn (ft)	1085
E.G. DS (ft)	1089.04	Culv Frctn Ls (ft)	24.7
W.S. DS (ft)	1088.96	Culv Exit Loss (ft)	2.2
Delta EG (ft)	28.04	Culv Entr Loss (ft)	1.14
Delta WS (ft)	27.98	Q Weir (cfs)	
E.G. IC (ft)	1105.03	Weir Sta Lft (ft)	
E.G. OC (ft)	1117.09	Weir Sta Rgt (ft)	
Culvert Control	Outlet	Weir Submerg	
Culv WS Inlet (ft)	1100.53	Weir Max Depth (ft)	
Culv WS Outlet (ft)	1088.96	Weir Avg Depth (ft)	
Culv Nml Depth (ft)	4	Weir Flow Area (sq ft)	
Culv Crt Depth (ft)	3.61	Min El Weir Flow (ft)	1119.01

Principal Spillway #1

Q Culv Group (cfs)	474.57	Culv Full Len (ft)	1103.55
# Barrels	1	Culv Vel US (ft/s)	24.17
Q Barrel (cfs)	474.57	Culv Vel DS (ft/s)	24.17
E.G. US. (ft)	1047.09	Culv Inv El Up (ft)	1059.67
W.S. US. (ft)	1086.2	Culv Inv El Dn (ft)	1028.17
E.G. DS (ft)	1031.61	Culv Frctn Ls (ft)	31.5
W.S. DS (ft)	1031.32	Culv Exit Loss (ft)	10.62
Delta EG (ft)	15.47	Culv Entr Loss (ft)	4.54
Delta WS (ft)	54.88	Q Weir (cfs)	
E.G. IC (ft)	1086.2	Weir Sta Lft (ft)	
E.G. OC (ft)	1047.09	Weir Sta Rgt (ft)	
Culvert Control	Outlet	Weir Submerg	
Culv WS Inlet (ft)	1064.67	Weir Max Depth (ft)	
Culv WS Outlet (ft)	1033.17	Weir Avg Depth (ft)	
Culv Nml Depth (ft)	1.07	Weir Flow Area (sq ft)	
Culv Crt Depth (ft)	5	Min El Weir Flow (ft)	1089.99

Emergency Spillway #3

E.G. US. (ft)	1170.51	Weir Sta US (ft)	2.06
W.S. US. (ft)	1170.5	Weir Sta DS (ft)	424.72
E.G. DS (ft)	1170.49	Weir Max Depth (ft)	5.26
W.S. DS (ft)	1170.3	Weir Avg Depth (ft)	5.03
Q US (cfs)	-1.38	Weir Submerg	0
Q Leaving Total (cfs)	12007.21	Min El Weir Flow (ft)	1165.23
Q DS (cfs)	-12009.2	Wr Top Wdth (ft)	422.66
Perc Q Leaving	868055.8	Q Gate Group (cfs)	
Q Weir (cfs)	12007.21	Gate Open Ht (ft)	
Q Gates (cfs)		Gate #Open	
Q Culv (cfs)		Gate Area (sq ft)	
Q Lat RC (cfs)		Gate Submerg	
Weir Flow Area (sq ft)	2124.79	Gate Invert (ft)	

Emergency Spillway #2

E.G. US. (ft)	1116.94	Weir Sta US (ft)	4.02
W.S. US. (ft)	1116.94	Weir Sta DS (ft)	369.18
E.G. DS (ft)	1116.94	Weir Max Depth (ft)	3.71
W.S. DS (ft)	1116.94	Weir Avg Depth (ft)	3.63
Q US (cfs)	-305.79	Weir Submerg	0
Q Leaving Total (cfs)	6856.14	Min El Weir Flow (ft)	1113.23
Q DS (cfs)	152.11	Wr Top Wdth (ft)	365.16
Perc Q Leaving	2242.08	Q Gate Group (cfs)	
Q Weir (cfs)	6856.13	Gate Open Ht (ft)	
Q Gates (cfs)		Gate #Open	
Q Culv (cfs)		Gate Area (sq ft)	
Q Lat RC (cfs)		Gate Submerg	
Weir Flow Area (sq ft)	1324.65	Gate Invert (ft)	

Emergency Spillway #1

E.G. US. (ft)	1086.2	Weir Sta US (ft)	7.46
W.S. US. (ft)	1086.19	Weir Sta DS (ft)	824.56
E.G. DS (ft)	1086.2	Weir Max Depth (ft)	4.23
W.S. DS (ft)	1086.2	Weir Avg Depth (ft)	4.18
Q US (cfs)	19371.3	Weir Submerg	0
Q Leaving Total (cfs)	18887.14	Min El Weir Flow (ft)	1081.97
Q DS (cfs)	474.57	Wr Top Wdth (ft)	817.1
Perc Q Leaving	97.5	Q Gate Group (cfs)	
Q Weir (cfs)	18887.14	Gate Open Ht (ft)	
Q Gates (cfs)		Gate #Open	
Q Culv (cfs)		Gate Area (sq ft)	
Q Lat RC (cfs)		Gate Submerg	
Weir Flow Area (sq ft)	3416.62	Gate Invert (ft)	

FRS#3 Dam Crest

E.G. US. (ft)	1170.49	Weir Sta US (ft)	
W.S. US. (ft)	1170.29	Weir Sta DS (ft)	
E.G. DS (ft)	1171.12	Weir Max Depth (ft)	
W.S. DS (ft)	1171.12	Weir Avg Depth (ft)	
Q US (cfs)	-12009.2	Weir Submerg	
Q Leaving Total (cfs)	0	Min El Weir Flow (ft)	1171.43
Q DS (cfs)	105.79	Wr Top Wdth (ft)	
Perc Q Leaving	0	Q Gate Group (cfs)	
Q Weir (cfs)		Gate Open Ht (ft)	
Q Gates (cfs)		Gate #Open	
Q Culv (cfs)		Gate Area (sq ft)	
Q Lat RC (cfs)		Gate Submerg	
Weir Flow Area (sq ft)		Gate Invert (ft)	

FRS#2 Dam Crest Upstream of Miller Road

E.G. US. (ft)	1119.54	Weir Sta US (ft)	0
W.S. US. (ft)	1119.54	Weir Sta DS (ft)	6014
E.G. DS (ft)	1119.54	Weir Max Depth (ft)	0.91
W.S. DS (ft)	1119.54	Weir Avg Depth (ft)	0.77
Q US (cfs)	2783.8	Weir Submerg	0
Q Leaving Total (cfs)	10970.7	Min El Weir Flow (ft)	1118.61
Q DS (cfs)	313.87	Wr Top Wdth (ft)	6014
Perc Q Leaving	394.09	Q Gate Group (cfs)	
Q Weir (cfs)	10970.7	Gate Open Ht (ft)	
Q Gates (cfs)		Gate #Open	
Q Culv (cfs)		Gate Area (sq ft)	
Q Lat RC (cfs)		Gate Submerg	
Weir Flow Area (sq ft)	4629.24	Gate Invert (ft)	

FRS#1 Dam Crest Upstream of Sun Valley Parkway

E.G. US. (ft)	1092.61	Weir Sta US (ft)	0
W.S. US. (ft)	1092.51	Weir Sta DS (ft)	12464.8
E.G. DS (ft)	1092.5	Weir Max Depth (ft)	1.3
W.S. DS (ft)	1092.5	Weir Avg Depth (ft)	0.82
Q US (cfs)	2778.35	Weir Submerg	0
Q Leaving Total (cfs)	25519	Min El Weir Flow (ft)	1091.2
Q DS (cfs)	-14048.2	Wr Top Wdth (ft)	12464.8
Perc Q Leaving	918.5	Q Gate Group (cfs)	
Q Weir (cfs)	25519	Gate Open Ht (ft)	
Q Gates (cfs)		Gate #Open	
Q Culv (cfs)		Gate Area (sq ft)	
Q Lat RC (cfs)		Gate Submerg	
Weir Flow Area (sq ft)	10221.9	Gate Invert (ft)	

Miller Road Culverts, FRS#2

Q Culv Group (cfs)	3.86	Culv Full Len (ft)	104
# Barrels	1	Culv Vel US (ft/s)	0.23
Q Barrel (cfs)	3.86	Culv Vel DS (ft/s)	0.23
E.G. US. (ft)	1116.95	Culv Inv El Up (ft)	1099.71
W.S. US. (ft)	1119.54	Culv Inv El Dn (ft)	1099.47
E.G. DS (ft)	1116.95	Culv Frctn Ls (ft)	0
W.S. DS (ft)	1116.95	Culv Exit Loss (ft)	0
Delta EG (ft)	0	Culv Entr Loss (ft)	0
Delta WS (ft)	2.59	Q Weir (cfs)	298.77
E.G. IC (ft)	1115.49	Weir Sta Lft (ft)	134.21
E.G. OC (ft)	1116.95	Weir Sta Rgt (ft)	784.96
Culvert Control	Outlet	Weir Submerg	1
Culv WS Inlet (ft)	1103.29	Weir Max Depth (ft)	4.95
Culv WS Outlet (ft)	1103.05	Weir Avg Depth (ft)	3.09
Culv Nml Depth (ft)		Weir Flow Area (sq ft)	2011.85
Culv Crt Depth (ft)	0.36	Min El Weir Flow (ft)	1112.01

Sun Valley Parkway Culvert, FRS#1

Q Culv Group (cfs)	315.71	Culv Full Len (ft)	100
# Barrels	4	Culv Vel US (ft/s)	0.79
Q Barrel (cfs)	78.93	Culv Vel DS (ft/s)	0.79
E.G. US. (ft)	1089.36	Culv Inv El Up (ft)	1067.53
W.S. US. (ft)	1092.5	Culv Inv El Dn (ft)	1066.94
E.G. DS (ft)	1089.35	Culv Frctn Ls (ft)	0
W.S. DS (ft)	1089.34	Culv Exit Loss (ft)	0.01
Delta EG (ft)	0.01	Culv Entr Loss (ft)	0
Delta WS (ft)	3.16	Q Weir (cfs)	4807.02
E.G. IC (ft)	1081.79	Weir Sta Lft (ft)	345.49
E.G. OC (ft)	1089.36	Weir Sta Rgt (ft)	2247.71
Culvert Control	Outlet	Weir Submerg	1
Culv WS Inlet (ft)	1077.53	Weir Max Depth (ft)	10
Culv WS Outlet (ft)	1076.94	Weir Avg Depth (ft)	5.66
Culv Nml Depth (ft)		Weir Flow Area (sq ft)	10759.3
Culv Crt Depth (ft)	1.25	Min El Weir Flow (ft)	1079.36

Johnson Road Culvert, FRS#1

Q Culv Group (cfs)	11.14	Culv Full Len (ft)	100
# Barrels	2	Culv Vel US (ft/s)	0.21
Q Barrel (cfs)	5.57	Culv Vel DS (ft/s)	0.21
E.G. US. (ft)	1087.16	Culv Inv El Up (ft)	1062.79
W.S. US. (ft)	1089.4	Culv Inv El Dn (ft)	1062.7
E.G. DS (ft)	1087.16	Culv Frctn Ls (ft)	0
W.S. DS (ft)	1087.16	Culv Exit Loss (ft)	0
Delta EG (ft)	0	Culv Entr Loss (ft)	0
Delta WS (ft)	2.24	Q Weir (cfs)	1619.93
E.G. IC (ft)	1087.17	Weir Sta Lft (ft)	19.81
E.G. OC (ft)	1087.16	Weir Sta Rgt (ft)	3111.39
Culvert Control	Outlet	Weir Submerg	1
Culv WS Inlet (ft)	1067.79	Weir Max Depth (ft)	10.39
Culv WS Outlet (ft)	1067.7	Weir Avg Depth (ft)	7.07
Culv Nml Depth (ft)		Weir Flow Area (sq ft)	21862.3
Culv Crt Depth (ft)	0.37	Min El Weir Flow (ft)	1076.78

Table 5-8

The Buckeye FRS Structures Hydraulic Responses to the PMP Events (This study and Dames & Moore, 1990)

	Unit	Buckeye FRSs		
		FRS#1	FRS#2	FRS#3
6-hour PMP				
Max. WSEL	ft	1092.5[a]	1119.5	1170.9
		1093.1[b]	1119.2	1172.1
Max. Flow on Dam Crest	cfs	25519.0	10970.7	0.0
		N/A[*]	N/A	N/A
Max. Water Depth on Dam Crest	ft	1.3	0.9	0.0
		1.9	0.6	0.7
Duration of Dam Overtopping	hr	5.0	4.3	0.0
		3.7	0.8	0.8
Max. Flow on Emergency Spillway	cfs	18887.1	6856.1	12007.2
		N/A	N/A	N/A
Max. Water Depth on Emergency Spillway	ft	4.2	3.7	5.3
		N/A	N/A	N/A
Duration of Emergency Spillway Overtopping	hr	75.0	58.0	14.0
		N/A	N/A	N/A
72-hour PMP				
Max. WSEL	ft	1092.0	1119.0	1168.0
		1091.9	1117.4	1170.0
Max. Flow on Dam Crest	cfs	6183.0	1641.6	0.0
		N/A	N/A	N/A
Max. Water Depth on Dam Crest	ft	0.8	0.4	0.0
		0.7	0.0	0.0
Duration of Dam Overtopping	hr	4.8	7.0	0.0
		4.0	0.0	0.0
Max. Flow on Emergency Spillway	cfs	11117.5	1523.1	4147.3
		N/A	N/A	N/A
Max. Water Depth on Emergency Spillway	ft	3.0	1.4	2.6
		N/A	N/A	N/A
Duration of Emergency Spillway Overtopping	hr	108	63.0	23.5
		N/A	N/A	N/A

[a] Results from PBS&J study.
[b] Results from Dames & Moore result (1990).
[*] N/A represents either "not applicable" or "not available from the report".

Table 5-9

Summary of Buckeye FRS Level Pool Routing Analyses

Item	Unit	FRS#1 (75.76 sq-mi)			FRS#2 (5.87 sq-mi)			FRS#3 (8.77 sq-mi)		
		PMP72hr	PMP6hr	100-yr	PMP72hr	PMP6hr	100-yr	PMP72hr	PMP6hr	100-yr
Peak Discharge	cfs	62,240	98,606	13,161	13,614	30,332	4,380	16,142	35,659	4,489
Time to Peak	hr	39.33	5.92	14.67	36.83	3.83	12.42	37	4.08	12.67
Runoff Volume	ac-ft	24,991	26,508	4,990	2,686	3,286	422	3,740	4,453	617
Runoff Volume	inch	6.20	6.50	1.24	8.65	10.60	1.36	8.00	9.50	1.32
Max WSEL	ft	1090.27	1090.78	1073.88	1119.23	1119.8	1108.8	1171.97	1172.4	1160.43
Max Depth on Dam	ft	0.37	0.88	0.00	0.23	0.80	0.00	0.17	0.60	0.00
Max Storage	ac-ft	22,988	23,851	3,504	1,839	1,954	344	2,846	2,964	520
Max Outflow	cfs	4,674	15,434	330	2,946	18,530	140	3,279	21,509	89
Duration of Overtopping	hr	12.67	10.25	0.00	6.75	3.83	0.00	10.25	4.33	0.00

Table 5-10

Comparison of FRS Level Pool Routing Analyses against Unsteady Hydraulic Models

		Level Pool Routing		Unsteady Hydraulic	
		Max Outflow	Max Depth on Dam Crest	Max Outflow	Max Depth on Dam Crest
FRS#3	PMP72hr	3,277	0.17	none	none
	PMP6hr	21,509	0.60	none	none
FRS#2					
	PMP72hr	2,946	0.23	1,641	0.40
	PMP6hr	18,530	0.80	10,970	0.90
FRS#1					
	PMP72hr	4,674	0.37	6,183	0.80
	PMP6hr	15,434	0.88	25,519	1.30

Addendum

Addendum of Buckeye FRS PMF Report to ADWR

Per the meeting with ADWR on August 19, 2004, more research and analyses have been conducted to clarify issues regarding the Buckeye Flood Retarding Structures (FRS) hydrology and hydraulics. The following summarizes the main concerns raised and the results of the additional analyses.

1. Concentrated vs. uniform lateral inflows in HEC-RAS unsteady model

ADWR questioned if the lateral inflows into the floodpool should be evenly distributed, and what the impact of different inflow distributions (concentrated or uniform) would be on the hydraulic results. Concentrated lateral inflows were used in the HEC-RAS unsteady models. A limited comparison between concentrated and uniform lateral inflows was conducted for FRS#2 to evaluate possible impacts to the hydraulic results. There are 3 lateral inflows into FRS#2. The unsteady flow data were modified by selecting the "uniform lateral inflow" option so that the inflow was evenly distributed between cross sections. The results indicated no change of the maximum water surface elevation (WSEL) for the entire reach as compared to selecting the "concentrated lateral inflows" option.

2. FRS capacity in terms of PMF percentages

ADWR would like to know what percentages of PMF the FRS can contain without any overtopping on the dam crest. A trial-and-error process was used to obtain FRS capacity in terms of PMF percentages in HEC-RAS unsteady models. As discussed at the meeting with ADWR, the calculation process stopped where there was no overtopping at the lowest spot along the FRS dam crests. However, this did not necessarily account for floodway overtopping. As a comparison, multiple ratios of the PMF (JR record) were used in HEC-1 level pool routing models to determine the percentages of the PMF that the FRS can contain without dam crest overtopping. The same inflow hydrographs were applied in both the HEC-RAS unsteady model and HEC-1 level pool routing model for each FRS. The results are presented in Table 1. As a reference, Dames & Moore (1990) results are presented in the last column. Consistently, the revised HEC-RAS unsteady model indicates a lower capacity for each FRS.

Table 1 FRS Capacity in PMF Percentages

	PBS&J		Dames & Moore
	HEC-RAS Unsteady Model	Level Pool Routing	Level Pool Routing
FRS#3	100%	100%	39%
FRS#2	62%	97%	47%
FRS#1	63%	100%	37%

3. Modifications of PBS&J level pool routing model

The results presented in Table 1 for the PMF percentages based on level pool routing are different from those given in the draft report previously submitted to ADWR. While reviewing the model results in conjunction with developing the additional data requested by ADWR, some discrepancies in the previous HEC-1 model results were noted as described below:

- The PBS&J level pool routing model accounted for the principal spillway discharge with a series of rating curves provided by the District for the PMF event. These rating curves were input as SQ records in the routing module. In addition to the SQ records for the principal spillway, ST records for flow overtopping the dam crest, SS records for the emergency spillway, and SV records for storage volume were input into the model. Upon further inspection, we noted that the HEC-1 model did not properly calculate the storage area at the emergency spillway crest elevation even though it showed a "`NORMAL END OF HEC-1`" message at the end of the run due to the principal spillway rating curve being limited to the emergency spillway crest elevation. The revised HEC-1 model eliminates the principal spillway rating curve, equivalent to assuming the primary outlets were plugged, which is the same method used by Dames & Moore. As the principal spillway peak discharge is relatively minor compared to flow through the emergency spillway and there is a potential for the primary spillway to become clogged with debris, this is a conservative and prudent assumption withiin the modeling. The differences of HEC-1 models between Dames & Moore and PBS&J are explained in the next section.

4. Comparison of PBS&J level pool routing models to Dames & Moore's

Key results are listed in Table 2 including peak inflows and outflows, time to peak, and ratios of peak inflow to outflow from both level pool routing models.

Table 2 Summary of Llevel Pool Routing Results

	FRS#1		FRS#2		FRS#3	
	PBSJ	D&M	PBSJ	D&M	PBSJ	D&M
Peak inflow (cfs)	98,606	144,648	30,332	42,243	35,659	54,275
Peak outflow (cfs)	33,448	137,708	15,005	28,203	15,742	45,452
Ratio of peak outflow/inflow (%)	34	95	49	67	44	84
Time to peak inflow (hr)	5.92	7.17	3.83	3.50	4.08	4.17
Time to peak outflow (hr)	7.58	7.50	4.50	3.92	4.92	4.42
Inflow/outflow peak offset (min)	100	20	40	25	50	15

Table 2 indicates the following differences between the two level pool routing studies:

1) Peak inflows from PBS&J study are 66~72% of those from Dames & Moore's study. The reasons were stated in the PMF report. Basically, Dames & Moore used higher precipitation depths without considering areal reduction, and applied high curve numbers (CN) in rainfall loss calculations. PBS&J used average precipitation depths based upon the entire contributing watershed to each FRS, and calculated the rainfall loss using the Green-Ampt method with the latest soil survey information.

2) PBS&J peak outflows after routing are on average 43% of peak inflows versus Dames & Moore peak outflows, which are on average 82% of peak inflows. Both

studies used the same shape of stage-volume curves. The reason may lie in offset of time to inflow/outflow peaks. The longer offset between the two peaks, the more the outflow peak tends to be attenuated. There are substantially longer offset times to inflow/outflow peaks from PBS&J study than from Dames & Moore's (last row in Table 2).

5. Comparison of peak inflow/outflow from other HEC-1 studies

There are other hydrology studies available for the Buckeye FRS system. Table 3 summarizes the peak inflow/outflow from other studies performed by the SCS (original design), the U.S. Army Corps of Engineer's (COE) in a recent inspection (FRS#1 only), Dames & Moore, and PBS&J. Programs and rainfall loss methods used for the analysis are indicated, too. A brief comparison is summarized as following.

1) All studies except PBS&J's used SCS-CN rainfall loss method.
2) All studies except Dames & Moore's considered areal reduction.
3) The major difference between the SCS original design and COE inspection is that the SCS design considered transmittal loss.

Table 3 Summary of Hydrology Studies for the Buckeye FRS System

	SCS	COE	D&M	PBS&J
Year Conducted	1974	1979	1990	2004
Rainfall Loss Method	SCS-CN in TR-20	SCS-CN in TR-20	SCS-CN in HEC-1	Green & Ampt in HEC-1
			FRS#1	
Peak Inflow Rate (cfs)	96,657	124,259	144,648	98,606
Peak Outflow Rate (cfs)	38,991	62,398	137,708	33,448
PMF Capacity	100%	65%	37%	100%
			FRS#2	
Peak Inflow Rate (cfs)	17,156		42,243	30,332
Peak Outflow Rate (cfs)	10,133		28,203	15,005
PMF Capacity	100%		47%	97%
			FRS#3	
Peak Inflow Rate (cfs)	20,669		54,275	35,659
Peak Outflow Rate (cfs)	12,966		45,452	15,742
PMF Capacity	100%		39%	100%

Results of the PBS&J 2004 study are simlar to the results of the 1974 SCS studies. Differences between the PBS&J study and the D&M study results are highlighted in Item 4 above.

Appendices

A. References

A.1 Data Collection Summary

The Data Collection Report was submitted on January 5, 2004. The following is the executive summary.

The Flood Control District of Maricopa County (District) contracted with PBS&J to perform the Buckeye/Sun Valley Area Drainage Master Study (ADMS). The study will identify area flooding, drainage, erosion, and sediment problems in the study area; evaluate existing floodplain delineations and delineate addition floodplains; identify a range of implementable solutions; and develop preliminary development guidelines for the area. The data collection task is the first step in this effort, and is the foundation for subsequent detailed studies.

Available mapping was compiled and an inventory is included in this report. A map exhibit prepared for this study includes the boundaries of the study area, known flooding problems, land ownership, and existing and future land uses.

Existing infrastructure and major utilities within the study area were identified and will be integrated into the hydrologic study for each sub-watershed.

Historic flooding events were investigated and information was collected from various sources. Historic flooding descriptions and photos are presented in this report. The log of drainage and flooding complaints maintained by the District was examined. The complaints originating from the study area were reviewed and are summarized in this report. The sites experiencing repetitive flooding will be emphasized in the study.

Previous hydrologic and hydraulic studies related to this study were reviewed and discussed. The studies that could be used as a basis for hydrology in the Buckeye/Sun Valley ADMS were identified. The studies are reviewed in sufficient detail to provide a basis for determining whether updating elements of those studies is feasible and adequate for the purposes of this project.

A.2 Referenced Documents

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Thomas, B.W., Hjalmarson, H.W. 1991. Estimated Manning's Roughness Coefficients for Stream Channels and Flood Plains in Maricopa County, Arizona. *Water Resource Diversion, USGS, Prepared for the Flood Control District of Maricopa County.*

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Zhao, Bing. March 17, 2003. Hydrologic Study for Bonita Dam, Maricopa County, Arizona. *Prepared for the Flood Control District of Maricopa County.*

B. General Documentation & Correspondence

The general documentation and correspondence for this project are organized in the Project Administration Report for the Buckeye/Sun Valley ADMS included in Volume VIII, Supporting Documents. The following sections are included in the Project Administration Report.

B.1 Special Problem Reports

B.2 Contact (Telephone) Reports

B.3 Meeting Minutes or Reports

B.4 General Correspondence

B.5 Contract Documents

Responses to NRCS/ADWR Review comments are included in Appendix B.6.

B.6 Responses to NRCS/ADWR Review

B.6 Responses to NRCS/ADWR Review

OVER-ALL NO.	DWG, SHT, PAGE NO.	ITEM NO.	COMMENT	RESPONSE	COMMENTS
NRCS - James Moore, PE					
1	General Comments	1	The total drainage area in the report was larger than the drainage area in Table 3. The drainage area was slightly larger for Sites 1 and 2 and slightly smaller for Site 3. No changes recommended.	A	No change made
2	General Comments	2	The storage capacities in the report for the flood water retarding structures varied from the storage capacities shown in Table 3. No changes recommended.	B	No change made
3	General Comments	3	The design rainfalls and corresponding runoff volumes in the report varied from the rainfall and runoff volumes in Table 3. The worksheets from Hydrometeorological Report No. 49 (HMR-49) document the PMP rainfall amounts used in the analysis. The runoff volumes varied due to change in rainfall amounts and the Green-Ampt procedure used to determine the rainfall losses. No changes recommended.	A	No change made
4	General Comments	4	The flood water retarding structures were design to include aerated sediment storage. The sediment rates for the watershed should be estimated. The required sediment storage should be determined for the design life of the structures. NRCS requires the storage routings start at the sediment storage elevation or the water surface after a 10-day drawdown. The three sites meet the 10-day drawdown requirement. Therefore, the level-pool routings should begin at the estimated sediment storage elevation.	D	The ADMS will include an estimate of watershed sediment yield; however, it is not reasonable to assume that sediment will accumulate to the normal floodpool elevation. Data from the Flood Control District of Maricopa County (FCD) Operations and Dam Safety Monitoring Programs indicates minimal sediments have accumulated within the FRS floodpool after 30 year's operation. Therefore, both the unsteady model and the level-pool routing models were developed using the ground terrain data.

OVER-ALL NO.	DWG, SHT, PAGE NO.	ITEM NO.	COMMENT	RESPONSE	COMMENTS
5	General Comments	5	The rainfall distribution on the PMP worksheets used a mix of the HMR-49 distributions identified as HMR No. 5 and EM1110-2-1411 in Table 4.7. Recommend correcting the time sequence of the incremental PMP and using the HMR No. 5 distribution.	D	The PMP worksheets were mainly used to calculate the rainfall depths in a 6- and 72-hour PMP. The PMP rainfall distribution patterns followed Dames & Moore (1990) as coded in the HEC-1 models. See more details in Comment #6.
6	General Comments	6	The rainfall distributions used in the HEC-1 analysis do not match the distributions from HMR-49 or PMP worksheets. The maximum one-hour rainfall can significantly affect the peak discharges of the basins. The distribution in the HEC-1 analysis resulted in 6.85 inches or rainfall in the maximum one-hour compared to 6.3 inches from HMR-49 for Site 1. The distribution in the HEC-1 analysis resulted in 9.51 inches or rainfall in the maximum one-hour compared to 10.5 inches from HMR-49 for Site 2. The distribution in the HEC-1 analysis resulted in 8.95 inches or rainfall in the maximum one-hour compared to 9.7 inches from HMR-49 for Site 1. Recommend considering a distribution closer to the distribution of HMR No. 5 in Table 4.7 of HMR-49.	D	The HEC-1 computer models require the PMP distribution to be in 5-min intervals due to relatively small time of concentration of the sub-basins. HMR-49 lists only the 6-hour PMP distribution for 1-hour intervals. To transform the 1-hr into a 5-min interval distribution, the method as developed in "Phase I Report: Project Calculations - Hydrologic Analysis - Buckeye Floodwater Retarding Structures #1, #2, and #3" by Dames & Moore (1990) was followed. Basically the method used EM1110-2-1411 distribution pattern.
7	General Comments	7	Each of the principal spillways have significant fall from the inlet to the outlet. Tail-water has minor effects on the discharge of the principal spillway (PS). The PS discharge does not have significant effects on the maximum water surface of the downstream reservoir. The weir flow equation may over-estimate the auxiliary spillway discharge.	A/B/C	Agree that tail-water has minimal effect on the "PS" discharge, which in turn has little effect on the maximum water surface elevation. The auxiliary spillway discharges for the three FRSs were calculated using weir equation applied according to the FCD standard procedures. The discharge coefficients were adapted from Table 2.4 in the "Phase I report: Hydrologic Analysis - Buckeye Floodwater Retarding Structures #1, #2, and #3" by Dames & Moore (1990).

OVER-ALL NO.	DWG, SHT, PAGE NO.	ITEM NO.	COMMENT	RESPONSE	COMMENTS
8	General Comments	8	The floodway overtopping should be used for the existing conditions analysis. If the Flood Control District proposes to increase the capacity of the floodway to contain the PMF, the future condition analysis could consider no overtopping of the floodway.	A	Several scenarios were modeled, including one with floodway overtopping. Floodway overtopping was not identified in any previous studies because the floodway overtopping was caused by backwater effect which is only available in the unsteady model.
9	General Comments	9	Section 5.3.2 discusses the expansion and contraction coefficients at culvert locations. The HECRAS model used the same coefficients for the cross-sections and culverts. Recommend changing the expansion and contraction coefficients for the culvert crossings.	A	Contraction and expansion coefficients for uniform conditions used 0.1 and 0.3, respectively; entrance and exit loss coefficients used 0.5 and 1.0 for culverts, respectively. Changes were made to the report to be consistent with the models.
10	General Comments	10	Many of the cross-sections encountered varying terrain. Some of the data points away from the main channel were adjusted to remove the presence of the gullies. The adjusted data reduced the potential flow/storage areas from the unsteady flow model. What effects would the additional area of the cross-sections have on the unsteady flow model?	B/C	Initially, existing terrain data was used to develop the unsteady models. However, this terrain data with gullies et al created multiple different critical depths within a single cross section, which either caused the model to become unstable or led to incorrect interpolations of the water surface profile. The gullies were removed because of the above reasons. It is anticipated that for the PMF event, removal of the gullies will not create observable difference to the final model results (although some conveyance and storage losses will result).
11	Comments Site 1	1	The outlets for sub-basins Q and R drain into the floodway. For the Level Pool Routing, the hydrographs for sub-basins Q and R should routed through the floodway to the reservoir before combining with the remaining hydrographs.	D	Agree to the comment, however, the level pool routing is a simplistic method. It is reasonable to combine all the concentration points routing through the reservoir given by the size of sub-basin Q and R, and the distance to the floodpool.
12	Comments Site 1	2	The "n-value" for the Principal Spillway pipe used in the HECRAS unsteady flow model was 0.0012. Recommend correcting to 0.012 in the final analysis.	A	Changes made to the models. At the PMF event, no observable changes to the final results.

OVER-ALL NO.	DWG, SHT, PAGE NO.	ITEM NO.	COMMENT	RESPONSE	COMMENTS
13	Comments Site 1	3	The "n-value" for the Sun Valley Parkway culverts used in the HECRAS unsteady flow model was 0.0013. Recommend correcting to 0.013 in the final analysis.	A	Changes made to the models. At the PMF event, no observable changes to the final results.
14	Comments Site 1	4	HECRAS utilizes ineffective flow areas to model crossings. The unsteady flow model did not include ineffective flow areas at the crossings. What effects would the ineffective flow areas have on the model results?	B/C	There are ineffective areas created due to numerous existing gullies. These areas were removed them in consideration of model instability. However, it is envisioned that the ineffective flow areas will have little noticeable impact on the PMF results.
15	Comments Site 2	1	The dam crest elevation shown in Table 3 is 1120.0 feet. The design crest elevation from the Flood Control District survey before datum adjustment is 1117.0 feet.	D	All data presented in the model are based on NAVD 88. The crest elevation is consistent with current information provided by FCD.
16	Comments Site 2	2	The auxiliary spillway bottom width shown in Table 3 is 300 feet. The auxiliary spillway bottom width from the Flood Control District survey is 350 feet.	D	The model used 350 feet for the auxiliary spillway. The spillway width is consistent with current information provided by FCD
17	Comments Site 2	3	HECRAS utilizes ineffective flow areas to model crossings. The unsteady flow model did not include ineffective flow areas at the crossings. What effects would the ineffective flow areas have on the model results?	B/C	There are ineffective areas created due to numerous existing gullies. These areas were removed in consideration of model instability. However, it is envisioned that the ineffective flow areas will have little noticeable impact on the PMF results.
18	Comments Site 3	1	The dam crest elevation shown in Table 3 is 1172.0 feet. The design crest elevation from the Flood Control District survey is 1170.0 feet.	D	The model used 1170.0 feet for the crest. The crest elevation is consistent with current information provided by FCD.
19	Comments Site 3	2	HECRAS utilizes ineffective flow areas to model crossings. The unsteady flow model did not include ineffective flow areas at the crossings. What effects would the ineffective flow areas have on the model results?	B/C	There are ineffective areas created due to numerous existing gullies. These areas were removed in consideration of model instability. However, it is envisioned that the ineffective flow areas will have little noticeable impact on the PMF results.

OVER-ALL NO.	DWG, SHT, PAGE NO.	ITEM NO.	COMMENT	RESPONSE	COMMENTS
ADWR - Mike Johnson, Ph.D., P.E.					
20	Comments	1	Section 4.2.6.1 – Rainfall Losses: The 72-hr general PMF should assume saturated antecedent moisture conditions. However, this change would not be expected to result in the 72-hr PMF becoming critical.	A	No change made
21	Comments	2	Section 4.2.6.3 – Lag Time: The USBR Hydrology Manual (Cudworth, 1989) suggests assigning Kn values for the PMF at the low end of reasonable values of the watershed conditions present. I have seen other studies reduce the 100-yr Kn by 20 percent. The Kn values for the current study should be reduced in some rational way for the PMF events.	B/C	We agree that the Kn value decreases as the storm becomes more severe and we documented this finding in the report as well. However, there is no guideline for a quantitative change of the Kn value in response to storm return frequencies. Therefore, the Kn values used in the model were based on field investigation and engineering judgments. This will be re-evaluated as part of the FRS1 Dam Rehabilitation Modeling effort currently underway as part of a separate contract.
22	Comments	3	Section 5.1 – Method Description: Table 5-1 gives pertinent elevation data in the NGVD29 datum. Because the modeling uses a different datum, these elevations should also be given in NAVD88.	A	The NGVD29 data in Table 5.1 were extracted from the original design TDN (1974). For a complete comparison, an additional column indicating the value in the NAVD88 datum will be added.

OVER-ALL NO.	DWG, SHT, PAGE NO.	ITEM NO.	COMMENT	RESPONSE	COMMENTS
23	Comments	4	Section 5.5.3 – Dam Crest and Emergency Spillway: The three emergency spillways are not traditional broad-crested weirs and therefore selection of an appropriate weir coefficient is difficult. The weir coefficients used in the current study are probably conservatively low. Higher rating curves (or weir coefficients) could likely be justified by developing steady- state HEC-RAS models with close cross-section spacing (to approximate rapidly-varied flow conditions). Models for this purpose would be expected to extend far enough downstream of the crest to accurately predict flow depths in the outflow channel and far enough upstream to predict head losses in the approach channel.	B/C	The Emergency spillways discharge coefficients were adapted from Table 2.4 in "Phase I report: Hydrologic Analysis - Buckeye Floodwater Retarding Structures #1, #2, and #3" by Dames & Moore (1990). Discharge coefficients for the dam crests on overtopping were calculated using Hager's equation from "Lateral outflow over side weirs" (Hager, William H., 1987) as directed by FCD. The suggested approach may have some merit and will be discussed with FCD for possible inclusion in the Buckeye 1 Rehabilitation Study currently underway as part of a separate contract.
24	Comments	5	Section 5.5.3 – Dam Crest and Emergency Spillway: Overtopping of the roadways and FRS's can accurately be modeled using the weir equation. Trapezoidal-shaped weirs typically have discharge coefficients ranging 2.7 to 3.1. The 1978 FHWA document Hydraulics of Bridge Waterways provides a chart for estimating values.	D	The model used 2.6 as default in HEC-RAS. The roadway does not normally maintain a trapezoidal-shaped.
25	Comments	6	Table 5-9 – Summary of Level-Pool Routing: The max WSEL for the FRS #3 72-hr PMF is incorrectly shown as "1191.97."	A	The max WSEL for FRS#3 should be 1171.97. The typo will be corrected.
26	Comments	7	The report should be revised to include the content of the addendum.	A	Agreed

OVER-ALL NO.	DWG, SHT, PAGE NO.	ITEM NO.	COMMENT	RESPONSE	COMMENTS
27	Discussion	1	Further analysis and documentation of overtopping at Miller Road would be required in order for the Department to assume it washes out for purpose of evaluating the safe flood capacity of FRS No. 2. Information that would need to be provided includes: a. As-built cross-section of Miller Road, b. Characterization of erosion rate for road embankment soils, c. Depth, flow velocities, and duration of overtopping including tailwater submergence, and d. Time required for failure of the road embankment. If the Department were to assume that Miller Road washes out, then the FCDMC and the Department would need to monitor for and be aware of any future road improvements that would make washout less likely.	B/C	Miller Road is a dirt road and would be vulnerable to washing out in a PMF event. To further investigate this issue, the sub-tasks identified by ADWR must be completed. Those tasks are outside of the current study scope. We will discuss this issue with FCD. The purpose of our model was to identify problems and provide the basis for future mitigation actions.
28	Discussion	2	A segment of FRS No. 2 is still predicted to overtop in the event of washout of Miller Road (i.e. to a maximum depth of 0.2 ft as reported in Table 5-3). Would restoring the crest in this segment to the design elevation prevent the overtopping?	B/C	According to our model, to prevent the overtopping with Miller Rd. in place, the dam crests of FRS No. 2 and Floodway No. 3 must be raised an additional 0.71 ft and 1.86 ft respectively. Without Miller Rd., an additional 0.2 ft and 1.40 ft must be added to the upstream of the FRS No. 2 dam crest and Floodway No. 3, respectively.

A – Will Comply

B – Consultant to Evaluate

C – Review Agency to Evaluate

D – Will Not Comply

C. Survey Field Notes

C.1 Survey Field Notes for Aerial Mapping Control

N/A

C.2 Survey Field Notes for Hydrologic Modeling

C.2.1 Sun Valley Parkway Major Culverts

Note: culvert number follows Alpha Engineering Report (1996)

Picture.1
Culvert #1 looking downstream

Picture.2
Culvert #1 looking upstream

Picture.3
Culvert #2 looking downstream

Picture.4
Culvert #2 looking upstream

Picture.5
Culvert #3 looking downstream

Picture.6
Culvert #3 looking upstream

Picture.7
Culvert #5 inlet

Picture.8
Culvert #5 looking downstream

Picture.9
Culvert #5 looking downstream into culvert

Picture.10
Culvert #5 looking upstream

Picture.11
Culvert #6 looking downstream

Picture.12
Culvert #6 looking downstream into culvert

Picture.13
Culvert #7 looking downstream

Picture.14
Culvert #10A looking upstream

Picture.15
Culvert #11 looking downstream

Picture.16
Culvert #11 looking downstream into culvert

Picture.17
Culvert #11 looking downstream into culvert

Picture.18
Culvert #11 looking upstream

Picture.19
Culvert #11 looking upstream into culvert

Picture.20
Culvert #12 looking downstream

Picture.21
Culvert #12 looking upstream

Picture.22
Culvert #13 looking downstream

Picture.23
Culvert #13 looking upstream

Picture.24
Culvert #16 looking downstream

Picture.25
Culvert #16 looking upstream

Picture.26
Culvert #18 looking downstream

Picture.27
Culvert #18 looking upstream

Picture.28
Culvert #21 looking downstream

Picture.29
Culvert #21 looking upstream

Picture.30
Culvert #22 looking downstream

Picture.31
Culvert #22 looking downstream into culvert

Picture.32
Culvert #22 looking upstream

Picture.33
Culvert #24 looking downstream

Picture.34
Culvert #24 looking downstream into culvert

Picture.35
Culvert #24 looking upstream

Picture.36
Culvert #25 looking downstream

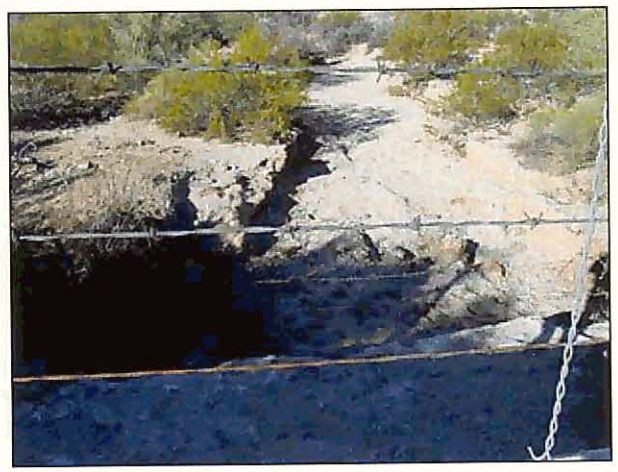
Picture.37
Culvert #25 looking downstream into culvert

Picture.38
Culvert #25 looking upstream

Picture.39
Culvert #35 looking downstream

Picture.40
Culvert #36 looking downstream

Picture.41
Culvert #36 looking upstream into culvert



Picture.42
Culvert #37 looking downstream

Picture.43
Culvert #37 looking downstream into culvert

Picture.44
Culvert #37 looking upstream

Picture.45
Culvert #38 looking downstream

Picture.46
Culvert #38 looking upstream

Picture.47
Culvert #38 looking upstream into culvert

C.2.2 Sub-basin Boundaries along Sun Valley Parkway

Picture.48

A2/A3 Boundary at Sun Valley Parkway looking east

Picture.49

C2/E2 Boundary at Sun Valley Parkway looking east

Picture.50

C2/E2 Boundary at Sun Valley Parkway looking west

Picture.51

E2/F3 Boundary at Sun Valley Parkway looking east

Picture.52

F3/H1 Boundary at Sun Valley Parkway looking south

Picture.53

F3/H1 Boundary at Sun Valley Parkway looking east

Picture.54

F3/H1 Boundary at Sun Valley Parkway looking north

Picture.55

H1/J1 Boundary at Sun Valley Parkway looking south

Picture.56

H1/J1 Boundary at Sun Valley Parkway looking east

Picture.57

H1/J1 Boundary at Sun Valley Parkway looking north

Picture.58

L2/M2 Boundary at Sun Valley Parkway looking south

Picture.59

L2/M2 Boundary at Sun Valley Parkway looking east

Picture.60

L2/M2 Boundary at Sun Valley Parkway looking north

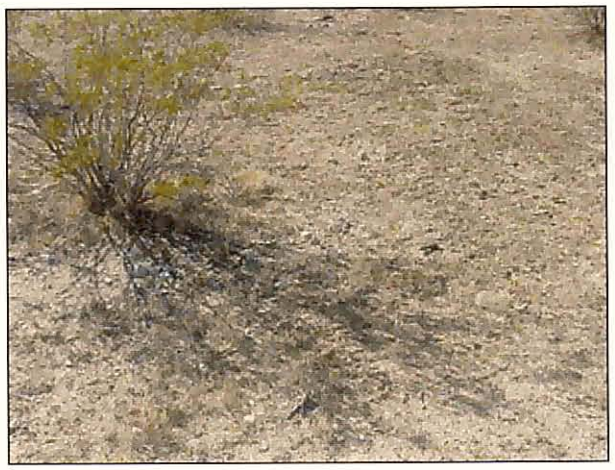
C.2.3 Routing Channels

Picture.61
Routing channel B1-B2

Picture.62
Routing channel B1-B2 (LOB)

Picture.63
Routing channel B1-B2 (ROB)

Picture.64
Routing channel J1-J2



Picture.65
Routing channel J1-J2 (LOB)

Picture.66
Routing channel J1-J2 (ROB)

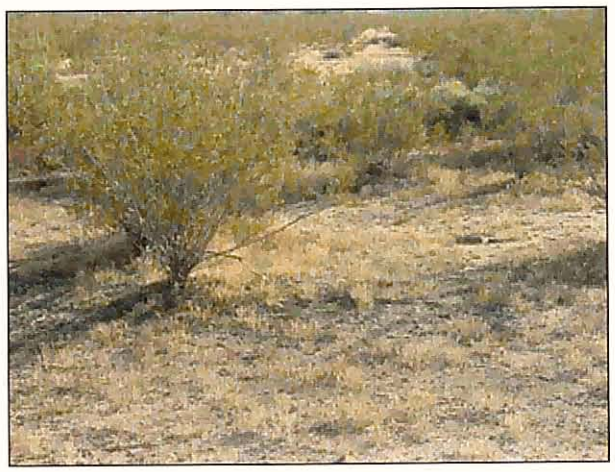
Picture.67
Routing channel L2-L3

Picture.68
Routing channel L2-L3 (LOB)

Picture.69
Routing channel L2-L3 (ROB)

Picture.70
Routing channel M1-M2

Picture.71
Routing channel M1-M2 (LOB)



Picture.72
Routing channel M1-M2 (ROB)

Picture.73
Routing channel W1-W2

Picture.74
Routing channel W1-W2 (LOB)

Picture.75
Routing channel W1-W2 (ROB)

C.3 Survey Field Notes for Hydraulic Modeling

C.3.1 Field Investigation by PBS&J

Picture 76
FRS #3 Principal Spillway Inlet

Picture 77
FRS #3 Floodway Looking West

Picture 78
FRS #3 Principal Spillway Outlet

Picture 79
FRS #3 Principal Spillway Outlet

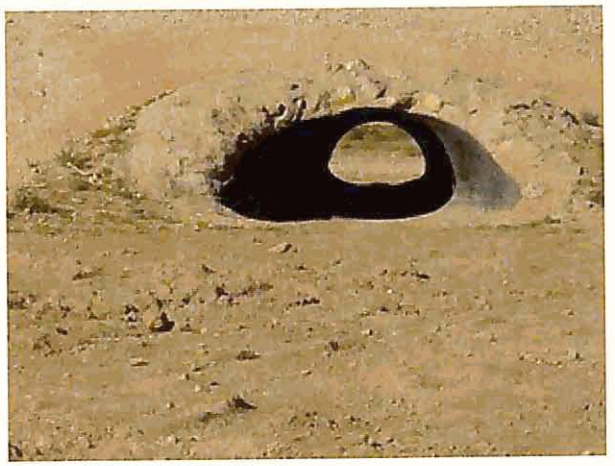
Picture 80
FRS #3 Principal Spillway Outlet

Picture 81
Baffle Chute at End of FRS #3 Floodway

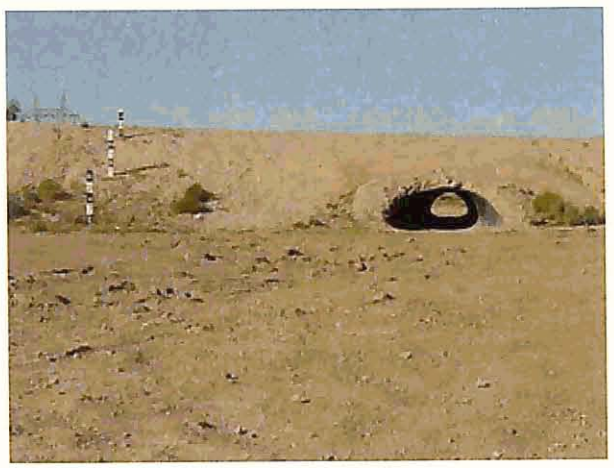
Picture 82
Baffle Chute at End of FRS #3 Floodway

Picture 83
FRS #3 Floodway Looking East

Picture 84
Beginning of FRS #2



Picture 85
Miller Road Culvert (FRS #2)



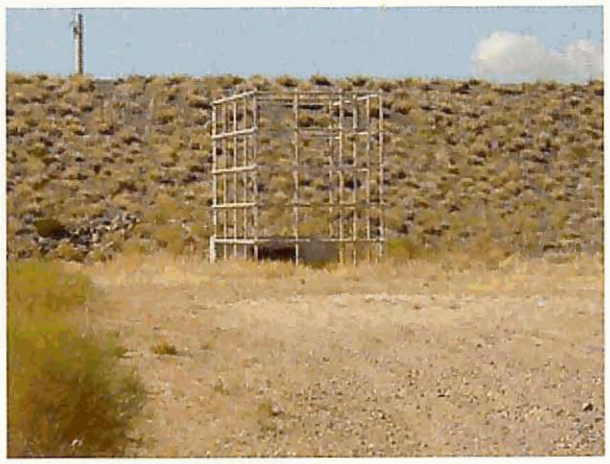
Picture 86
Miller Road Culvert (FRS #2)

Picture 87
FRS #2 Principal Spillway Outlet

Picture 88
FRS #2 Floodway Looking West

Picture 89
Johnson Road Culvert (FRS #1)

Picture 90
FRS #1 Principal Spillway Inlet



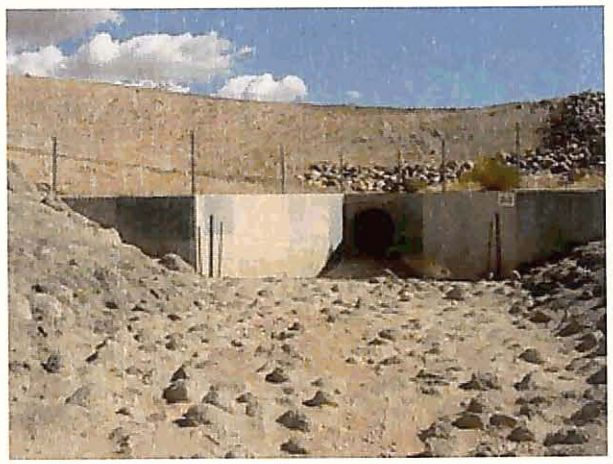
Picture 91
FRS #1 Principal Spillway Inlet

Picture 92
FRS #1 Emergency Spillway Looking East

Picture 93
FRS #1 Emergency Spillway Looking North

Picture 94
FRS #1 Floodway Looking West

Picture 95
FRS #1 Principal Spillway Outlet

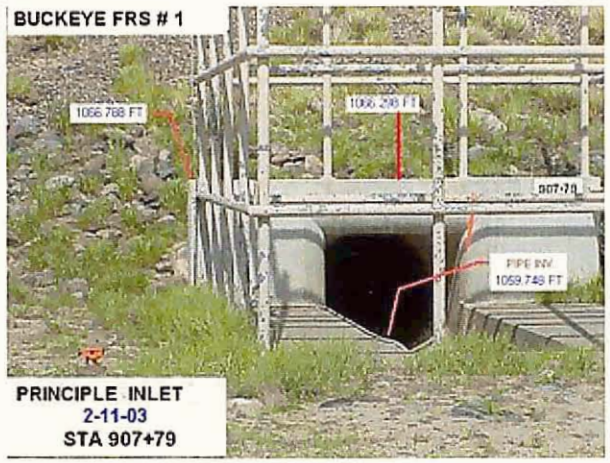


Picture 96
FRS #1 Principal Spillway Outlet

Picture 97
FRS #1 Principal Spillway Outlet

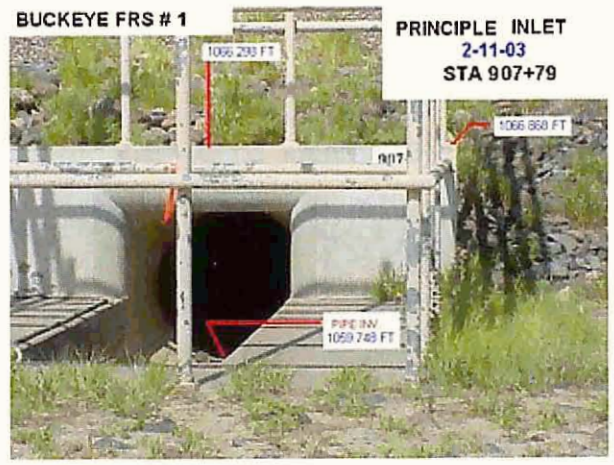
Picture 98
FRS #1 Principal Spillway Outlet

C.3.2 Field Investigation by the District (2002-2003)



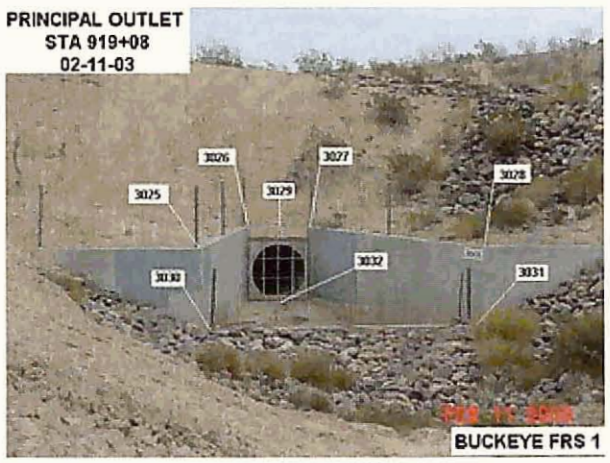
Picture 99

FRS #1 Principal Spillway Inlet



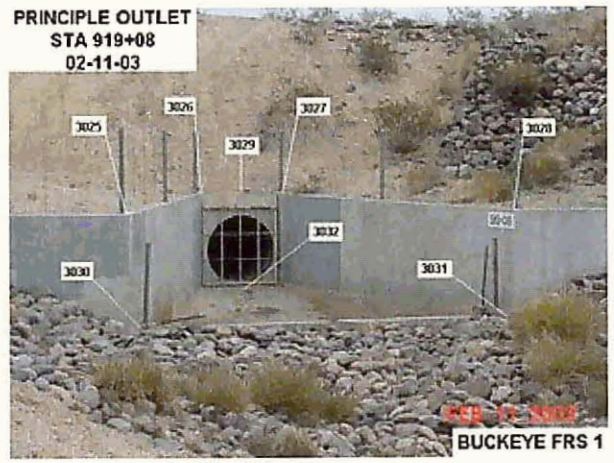
Picture 100

FRS #1 Principal Spillway Inlet



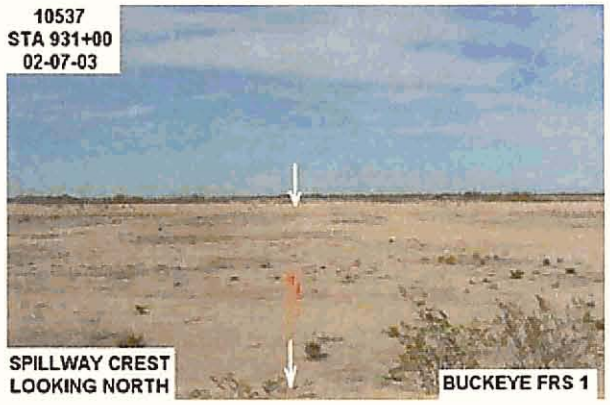
Picture 101

FRS #1 Principal Spillway Outlet



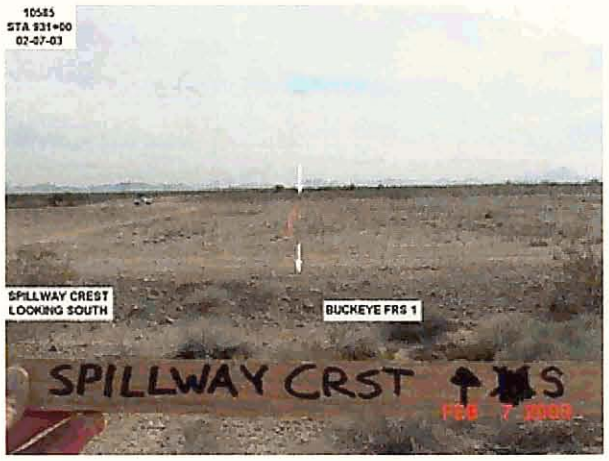
Picture 102

FRS #1 Principal Spillway Outlet

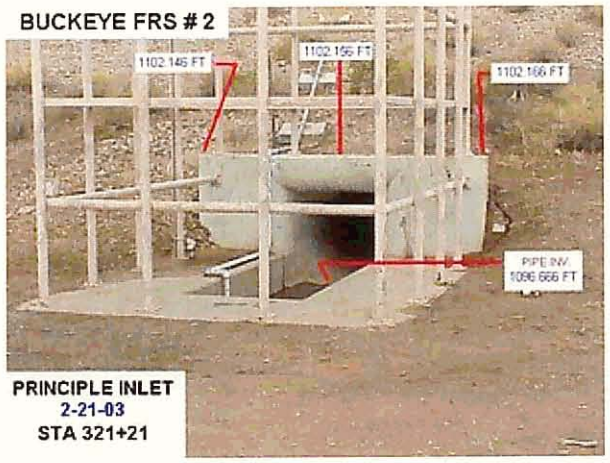


Picture 103
FRS #1 Emergency Spillway

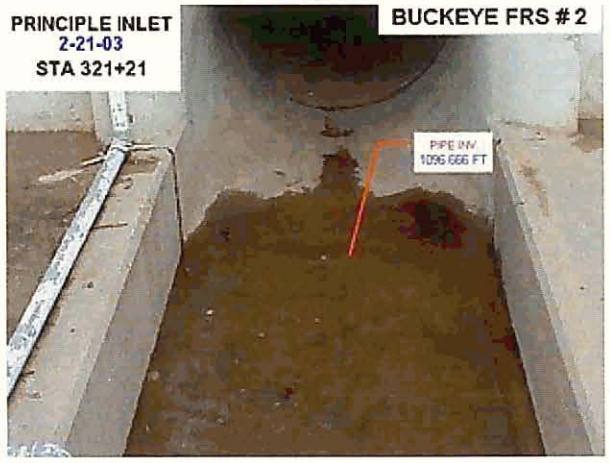
Picture 104
FRS #1 Emergency Spillway



Picture 105
FRS #1 Emergency Spillway



Picture 106
FRS #2 Principal Spillway Inlet



Picture 107
FRS #2 Principal Spillway Inlet

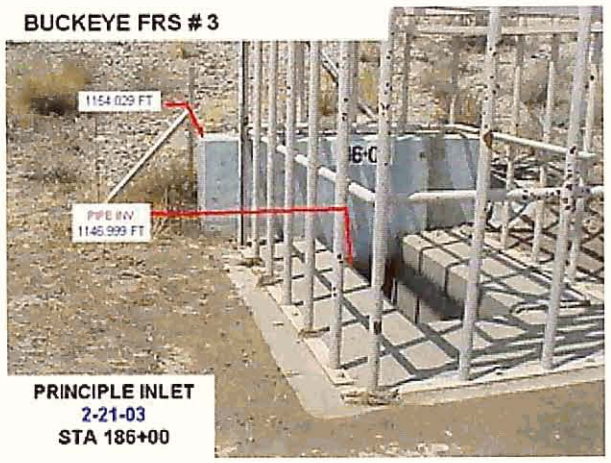
Picture 108
FRS #2 Principal Spillway Outlet

Picture 109

FRS #2 Emergency Spillway

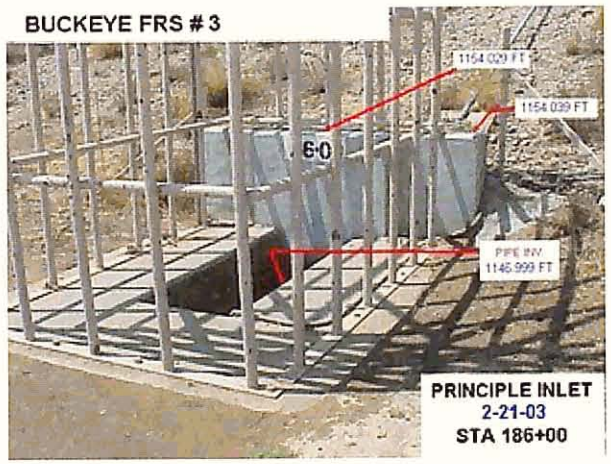
Picture 110

FRS #2 Emergency Spillway



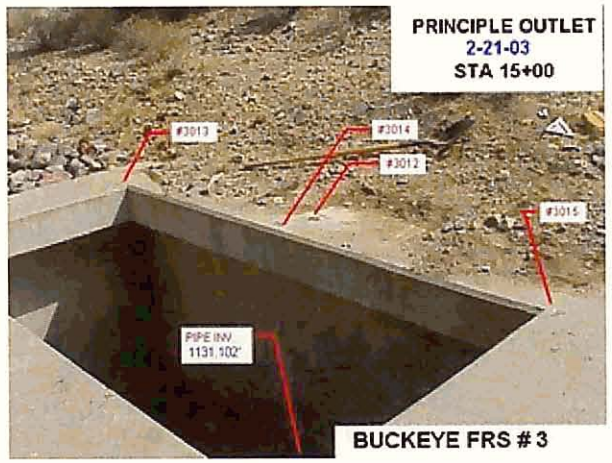
Picture 111

FRS #3 Principal Spillway Inlet



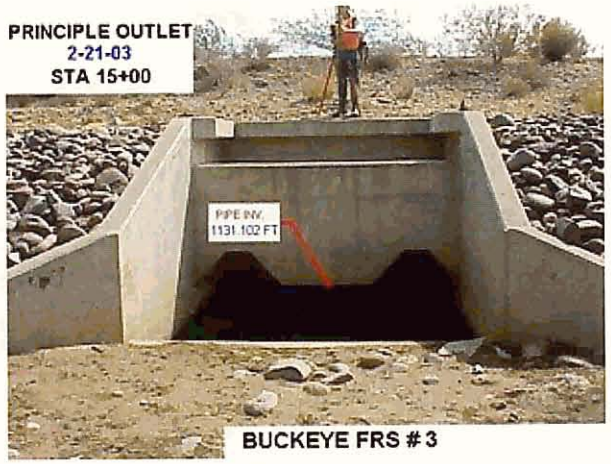
Picture 112

FRS #3 Principal Spillway Inlet



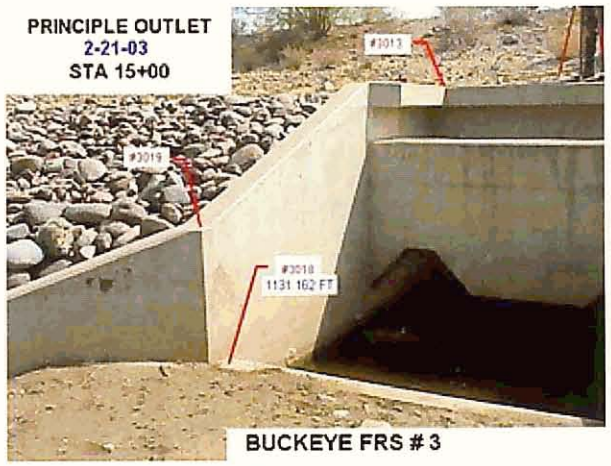
Picture 113

FRS #3 Principal Spillway Outlet



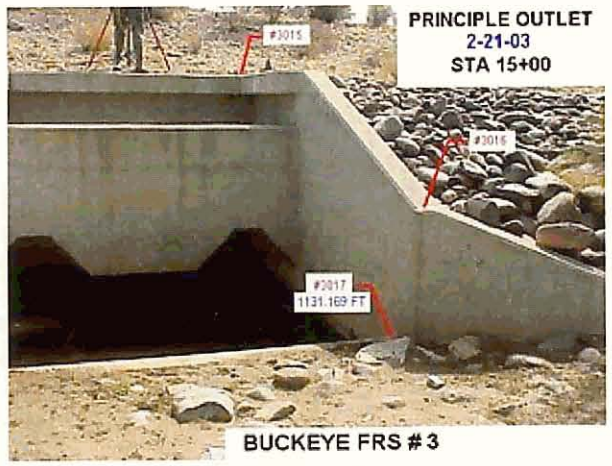
Picture 114

FRS #3 Principal Spillway Outlet



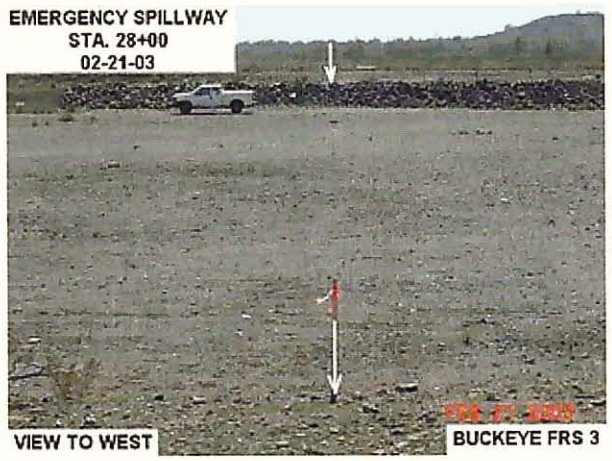
Picture 115

FRS #3 Principal Spillway Outlet



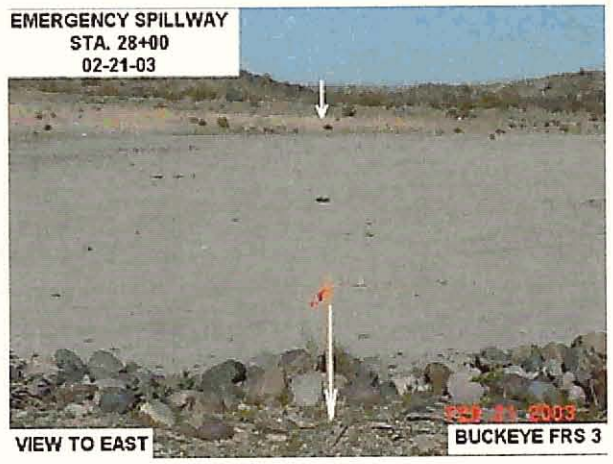
Picture 116

FRS #3 Principal Spillway Outlet



Picture 117

FRS #3 Emergency Spillway



Picture 118

FRS #3 Emergency Spillway

D. Hydrologic Analysis Supporting Documentation

D.1 Precipitation Data

Table D.1-1

Rainfall Amount Calculation from Prefre, in DDMSW (ver. 2.1)

	FRS 1 (Area=75.76 sq-mile)							
	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr
	2	5	10	25	50	100	200	500
5 MIN	0.34	0.43	0.49	0.58	0.65	0.72		
10 MIN	0.50	0.65	0.74	0.88	0.99	1.10		
15 MIN	0.61	0.81	0.94	1.12	1.27	1.41		
30 MIN	0.81	1.08	1.26	1.52	1.71	1.91		
1 HOUR	0.99	1.33	1.56	1.89	2.14	2.39		
2 HOUR	1.08	1.48	1.74	2.11	2.39	2.67		
3 HOUR	1.15	1.57	1.86	2.25	2.56	2.87		
6 HOUR	1.27	1.76	2.08	2.53	2.88	3.23		
12 HOUR	1.39	1.97	2.35	2.88	3.29	3.69		
24 HOUR	1.52	2.18	2.62	3.23	3.70	4.16	4.63	5.23
	FRS 2 (Area=5.82 sq-mile)							
	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr
	2	5	10	25	50	100	200	500
5 MIN	0.32	0.42	0.49	0.59	0.66	0.74		
10 MIN	0.48	0.64	0.75	0.90	1.02	1.13		
15 MIN	0.59	0.80	0.94	1.14	1.30	1.45		
30 MIN	0.77	1.07	1.27	1.54	1.76	1.97		
1 HOUR	0.94	1.32	1.57	1.92	2.19	2.46		
2 HOUR	1.03	1.46	1.74	2.14	2.44	2.74		
3 HOUR	1.09	1.55	1.86	2.28	2.60	2.93		
6 HOUR	1.20	1.72	2.07	2.55	2.91	3.28		
12 HOUR	1.31	1.91	2.31	2.86	3.29	3.70		
24 HOUR	1.42	2.11	2.56	3.18	3.66	4.13	4.61	5.22
	FRS 3 (Area=8.77 sq-mile)							
	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr	200-Yr	500-Yr
	2	5	10	25	50	100	200	500
5 MIN	0.32	0.42	0.49	0.58	0.66	0.73		
10 MIN	0.48	0.63	0.74	0.89	1.00	1.12		
15 MIN	0.58	0.79	0.93	1.13	1.28	1.43		
30 MIN	0.77	1.06	1.26	1.53	1.74	1.95		
1 HOUR	0.94	1.31	1.56	1.90	2.17	2.43		
2 HOUR	1.03	1.45	1.73	2.12	2.42	2.71		
3 HOUR	1.09	1.55	1.85	2.26	2.58	2.90		
6 HOUR	1.21	1.72	2.07	2.54	2.90	3.26		
12 HOUR	1.33	1.93	2.32	2.86	3.28	3.70		
24 HOUR	1.45	2.13	2.58	3.19	3.67	4.14	4.62	5.23

Table D.1-2

Summary of Rainfall Areal Reduction for 200 and 500-Year Storm Events

Area [mi^2]	Reduction Factor	FRS 1		FRS 2		FRS 3	
		200-yr 24-hr	500-yr 24-hr	200-yr 24-hr	500-yr 24-hr	200-yr 24-hr	500-yr 24-hr
0	1.000	4.63	5.23	4.61	5.22	4.62	5.23
10	0.950	4.40	4.97	23.00	25.99	120.31	135.90
20	0.918	4.25	4.80	22.23	25.11	116.26	131.33
30	0.900	4.17	4.71	21.79	24.62	113.98	128.75
40	0.887	4.11	4.64	21.48	24.26	112.33	126.89
50	0.877	4.06	4.59	21.24	23.99	111.07	125.46
60	0.870	4.03	4.55	21.07	23.80	110.18	124.46
70	0.863	4.00	4.51	20.90	23.61	109.29	123.46
80	0.857	3.97	4.48	20.75	23.44	108.53	122.60
90	0.852	3.94	4.46	20.63	23.30	107.90	121.88
100	0.848	3.93	4.44	20.53	23.20	107.39	121.31
110	0.845	3.91	4.42	20.46	23.11	107.01	120.88

Figure D.1-1a

Buckeye Area 3 FRS #1 IDF Graph

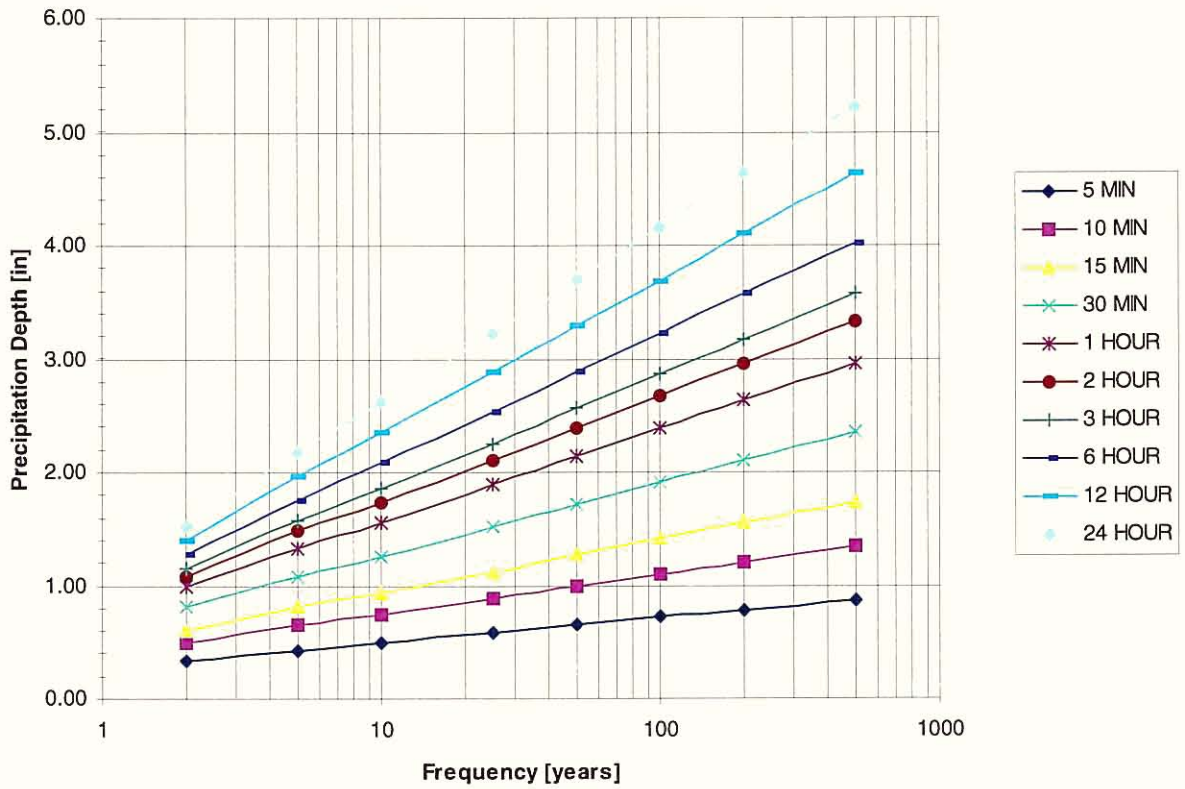


Figure D.1-1b

Buckeye Area 3 FRS #2 IDF Graph

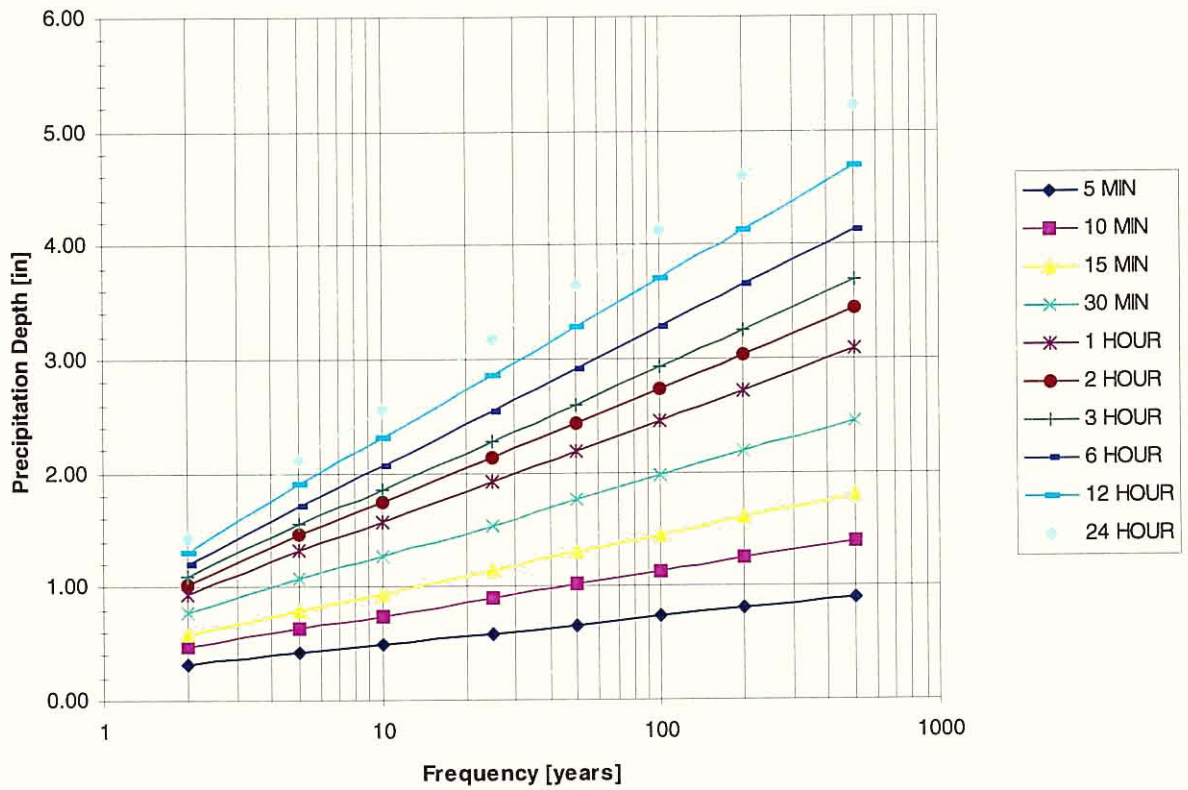


Figure D.1-1c

Buckeye Area 3 FRS #3 IDF Graph

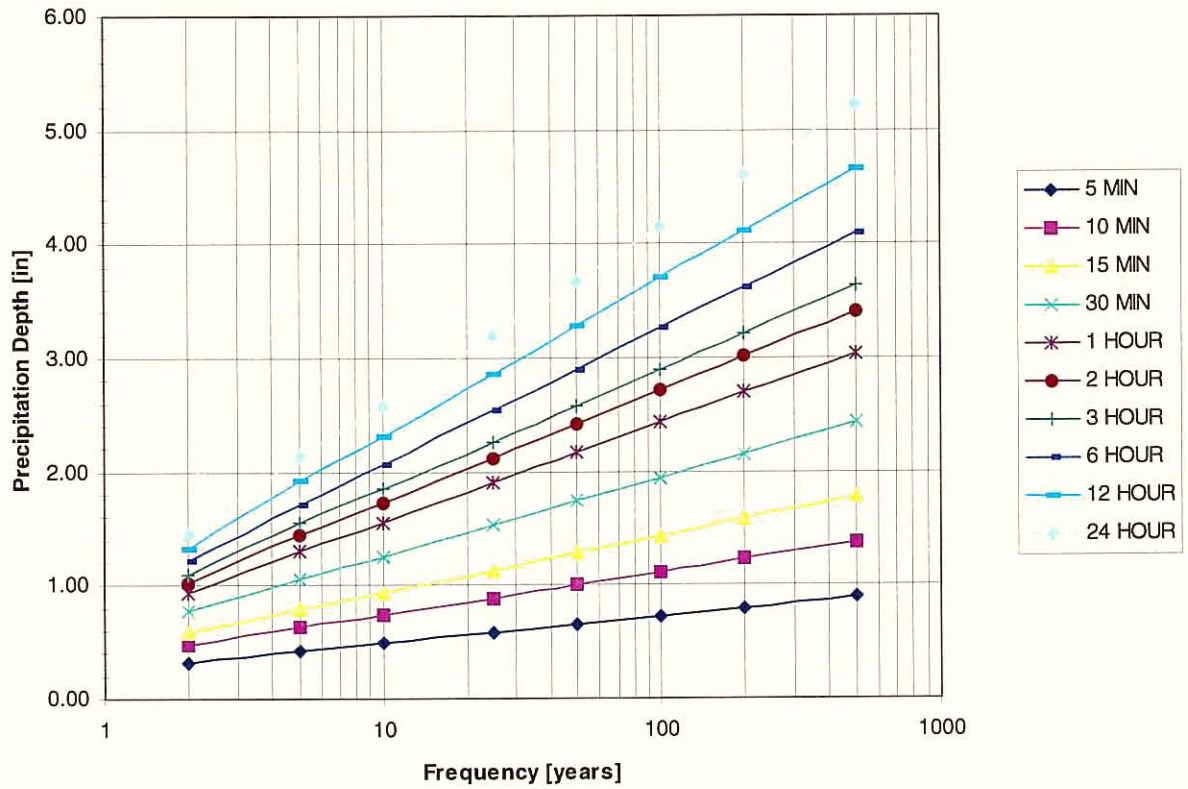


Figure D.1-2

Area 3 PMP 6-hour Incremental Rainfall Distribution

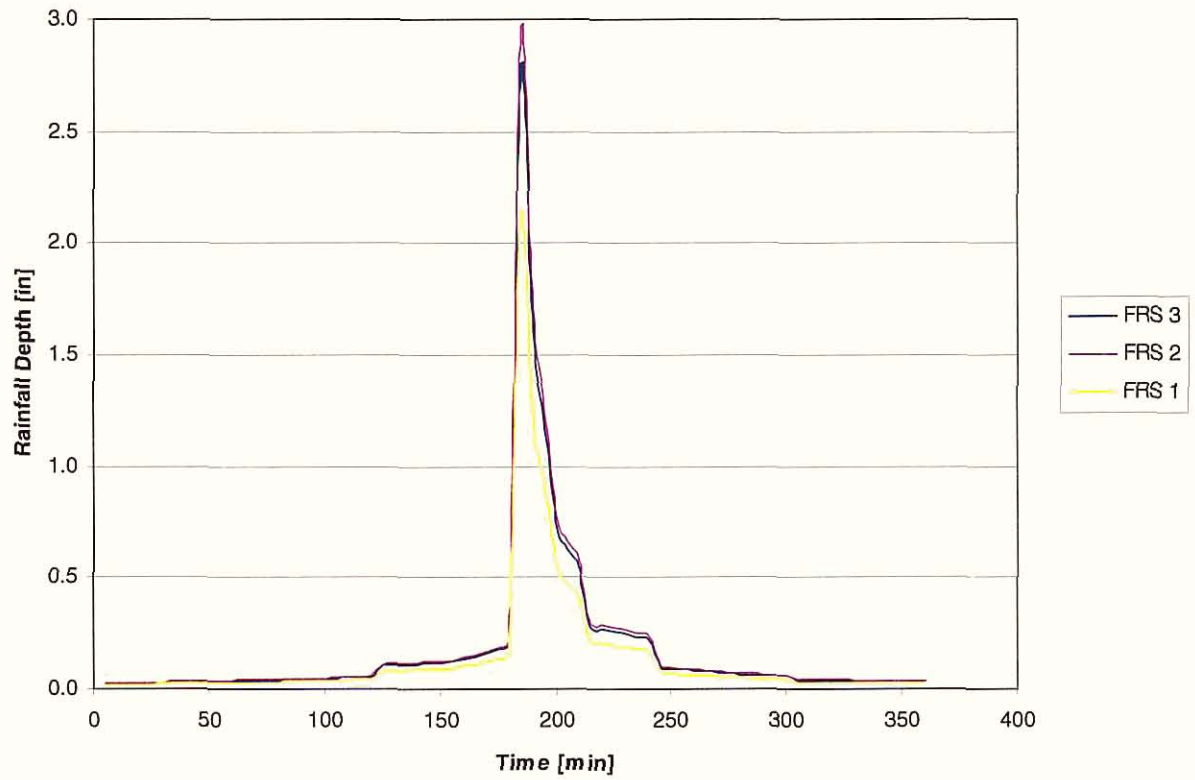
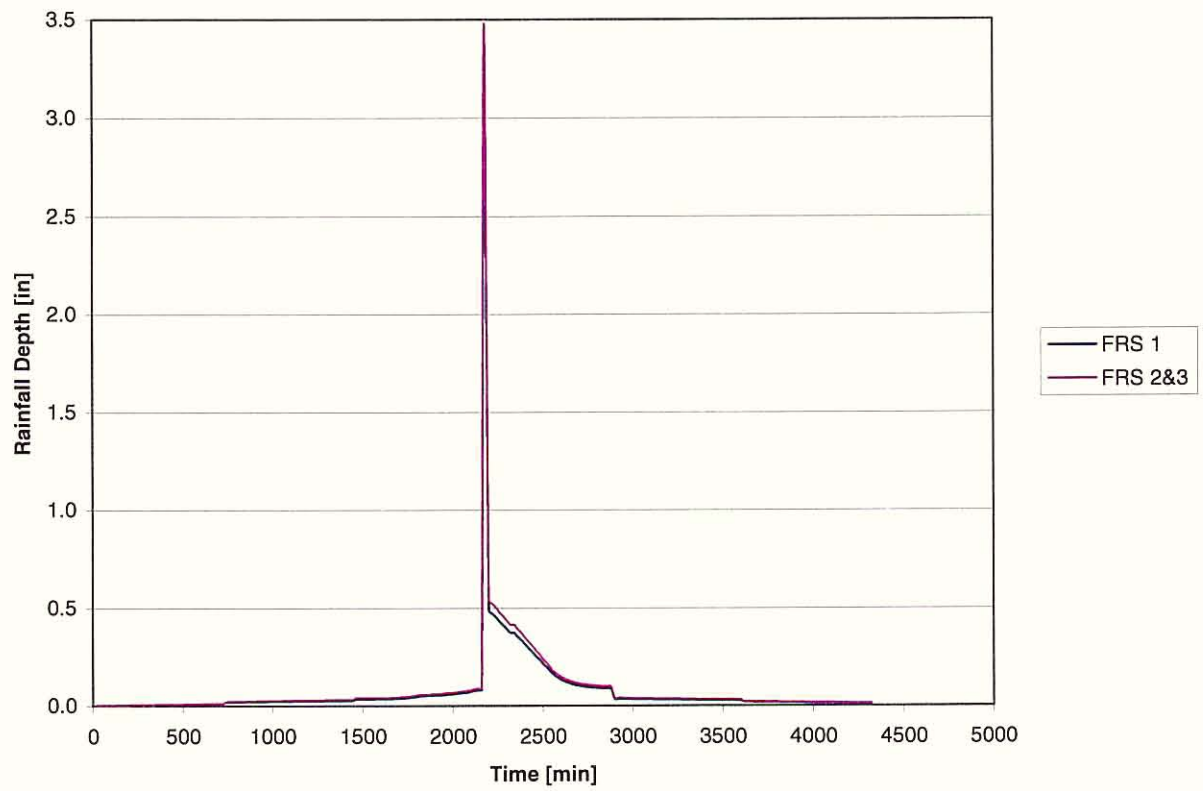


Figure D.1-3

Buckeye Area 3 PMP 72-hour Incremental Rainfall Distribution



D.2 Physical Parameter Calculations

Table D.2-1a

Existing Conditions Land Use Look-up Table for Area 3

LU Code	Land Use Description	IA	RTIMP	% Veg	Saturation
110	Rural Residential (<= 1/5 du per acre)	0.30	5	30	normal
120	Estate Residential (1/5 du per acre to 1 du per acre)	0.30	5	30	normal
130	Large Lot Residential - Single Family (1 du per acre to 2 du per acre)	0.30	15	50	normal
140	Medium Lot Residential - Single Family (2-4 du per acre)	0.25	30	50	normal
150	Small Lot Residential - Single Family (4-6 du per acre)	0.25	30	50	normal
160	Very Small Lot Residential - Single Family (>6 du per acre-includes mobile home	0.25	40	50	normal
170	Medium Density Residential - Muli Family (5-10 du per acre)	0.25	45	50	normal
180	High Density Residential - Multi Family (10-15 du per acre)	0.25	45	50	normal
190	Very High Density Residential - Multi Family (> 15 du per acre)	0.25	45	50	normal
200	General Commercial (Commercial where no detail available)	0.10	80	60	normal
210	Specialty Commercial (<=50,000 sq. ft.)	0.10	80	65	normal
220	Neighborhood Commercial (50,000 to 100,000 sq. ft.)	0.10	80	65	normal
230	Community Commercial (100,000 to 500,000 sq. ft.)	0.10	80	75	normal
240	Regional Commercial (500,000 to 1,000,000 sq. ft.)	0.10	80	65	normal
250	Super-Regional Commercial (>= 1,000,000 sq. ft.)	0.10	80	70	normal
300	General Industrial (Industrial where no detail available)	0.15	55	60	normal
310	Warehouse/Distribution Centers	0.10	80	75	normal
320	Industrial	0.15	55	60	normal
400	Office General (Office where no detail available)	0.10	80	75	normal
410	Office Low Rise (1-4 stories)	0.10	80	75	normal
420	Office Mid Rise (5-12 stories)	0.10	80	75	normal
430	Office High Rise (13 stories or more)	0.10	80	75	normal
510	Tourist and Visitor Accommodations (Hotels, motels and resorts)	0.10	80	75	normal
520	Educational (Public schools, private schools and universities)	0.29	45	80	normal
530	Institutional (Includes hospitals and churches)	0.10	80	75	normal
540	Cemeteries	0.10	5	90	normal
550	Public Facilities (Includes community centers, power sub-stations, libraries, ci	0.10	80	75	normal
560	Special Events (Includes stadiums, sports complexes and fairgrounds)	0.10	80	75	normal
570	Other Employment - low (Proving grounds and land fills)	0.10	80	75	normal
580	Other Employment - medium	0.10	80	75	normal
590	Other Employment - high	0.10	80	75	normal
600	General Transportation (Transportation where no detail available)	0.10	80	75	normal
610	Transportation (Includes railroads, railyards, transit centers and freeways)	0.10	80	75	normal
620	Airports (Includes public use airports)	0.15	55	60	normal
700	General Open Space (Open space where no detail available)	0.10	5	90	normal
710	Active Open Space (Includes parks)	0.10	5	90	normal

LU Code	Land Use Description	IA	RTIMP	% Veg	Saturation
720	Golf courses	0.10	5	90	normal
730	Passive Open Space (Includes mountain preserves and washes)	0.10	0	90	normal
740	Water	0.00	100	0	wet
750	Agriculture	0.50	0	85	normal
810	Business Park (Includes enclosed industrial, office or retail in a planned envir	0.10	80	75	normal
900	Vacant (Existing land use database only)	0.35	0	25	dry

Table D.2-1b

Future Conditions Land Use Look-up Table for Area 3

LU Code	Land Use Description	IA	RTIMP	% Veg	Saturation
100	Central, general, office, intermediate commerce	0.10	80	75	normal
200	Medium Density Residential (6,000-12,000 sq. ft lot size	0.25	30	50	normal
710	Active Open Space (Includes parks)	0.10	5	90	normal
900	Vacant (Existing land use database only)	0.35	0	25	dry

Table D.2-2

Soil Map look-up Table for Buckeye Area 3

Soil Code	Description	XKSAT	RTIMP	% Effective
1	Antho sandy loams	0.41	0	100
2	Antho gravelly sandy loams	0.41	0	100
3	Antho-Carrizo-Maripo complex	0.58	0	100
4	Antho-Carrizo-Maripo complex, low precipitation	0.58	0	100
10	Brios-Carrizo complex, 1 to 5 percent slopes	0.94	0	100
11	Brios-Carrizo complex, low precipitation, 1 to 5 percent slopes	0.94	0	100
14	Carrizo very gravelly sand	1.04	0	100
15	Carrizo-Gunsight complex, 1 to 5 percent slopes	0.54	0	100
19	Chuckawalla-Gunsight complex, 1 to 8 percent slopes	0.19	0	100
20	Chuckawalla-Gunsight complex, low precipitation, 1 to 8 percent slopes	0.19	0	100
21	Cipriano very gravelly loam	0.38	0	100
29	Denure-Momoli-Carrizo complex	0.34	0	100
30	Denure-Momoli-Carrizo complex, low precipitation	0.34	0	100
47	Ebon-Gunsight-Cipriano association, 3 to 25 percent slopes	0.11	0	100

Soil Code	Description	XKSAT	RTIMP	% Effective
48	Ebon-Pinamt complex, 3 to 20 percent slopes	0.06	0	100
49	Ebon-Pinamt complex, 20 to 40 percent slopes	0.06	0	100
52	Gachado-Lomitas-Rock outcrop complex, 7 to 55 percent slopes	0.16	20	100
55	Gilman loams	0.27	0	100
68	Gunsight-Cipriano complex, 1 to 7 percent slopes	0.63	0	100
69	Gunsight-Cipriano complex, low precipitation, 1 to 7 percent slopes	0.63	0	100
70	Gunsight-Rillito complex, 1 to 25 percent slopes	0.36	0	100
71	Gunsight-Rillito complex, low precipitation, 1 to 40 percent slopes	0.36	0	100
91	Momoli-Carrizo complex	0.93	0	100
92	Momoli-Carrizo complex, low precipitation	0.93	0	100
98	Pinamt-Tremant complex, 1 to 10 percent slopes	0.37	0	100
99	Pinamt-Tremant complex, low precipitation, 1 to 10 percent slopes	0.37	0	100
100	Quilotosa-Vaiva-Rock outcrop complex, 20 to 65 percent slopes	0.4	20	100
102	Rillito gravelly loam, 1 to 8 percent slopes	0.4	0	100
106	Sal-Cipriano complex, 1 to 10 percent slopes	0.18	0	100
107	Sal-Cipriano complex, low precipitation, 1 to 10 percent slopes	0.18	0	100
110	Suncity-Cipriano complex, 1 to 7 percent slopes	0.13	0	100
114	Tremant gravelly loams, low precipitation	0.39	0	100
123	Vaiva very gravelly loam, 1 to 20 percent slopes	0.37	0	100
999	Blank/Unknown/Uncertain	0	0	100
AL	Antho association	0.4	0	100
AM	Antho-Valencia association	0.39	0	100
CO	Cheriono-Rock outcrop complex	0.29	20	100
CV	Coolidge-Laveen association	0.39	0	100
GM	Gilman-Antho association	0.29	0	100
GN	Gilman-Laveen association	0.25	0	100
PT	Pinal gravelly loam	0.4	0	100
RS	Rock outcrop-Cherioni complex	0.4	65	100
TB	Torrifluvents	0.4	0	100
AGB	Antho-Carrizo complex, 0 to 3 percent slopes	0.4	0	100
GWD	Gunsight-Pinal complex, 1 to 10 percent slopes	0.35	0	100
GYD	Gunsight-Rillito complex, 0 to 10 percent slopes	0.26	0	100
HLC	Harqua-Gunsight complex, 0 to 5 percent slopes	0.14	0	100
LEV	Man-made levee	0	0	100
PRB	Perryville-Rillito complex, 0 to 3 percent slopes	0.28	0	100
PWB	Pinal-Suncity complex, 0 to 3 percent slopes	0.38	0	100
PYD	Pinamt-Tremant complex, 1 to 10 percent slopes	0.2	0	100
TSC	Tremant-Rillito complex, 0 to 5 percent slopes	0.14	0	100

Table D.2-3

Summary of Green & Ampt Parameters for Buckeye Area 3 Existing Conditions

Subbasin	Area [mi^2]	IA	DTHETA	PSIF	XKSAT	RTIMP
A1	1.11	0.350	0.350	4.058	0.417	9.677
A2	1.66	0.350	0.350	4.547	0.339	0.000
A3	3.73	0.350	0.350	4.230	0.405	0.000
B1	0.56	0.350	0.350	3.733	0.470	6.272
B2	0.49	0.350	0.350	3.617	0.513	0.000
B3	1.99	0.331	0.331	4.284	0.389	0.000
C1	3.59	0.215	0.296	3.933	0.407	21.409
C2	1.92	0.350	0.350	3.920	0.436	0.000
C3	3.79	0.345	0.345	4.357	0.372	0.000
C4	1.40	0.327	0.327	4.737	0.290	0.000
E1	0.96	0.350	0.378	5.377	0.222	0.690
E2	1.98	0.350	0.360	5.043	0.256	0.000
E3	2.49	0.350	0.350	4.358	0.347	0.000
E4	3.04	0.350	0.350	3.816	0.439	0.000
E5	1.31	0.347	0.344	3.940	0.429	0.968
E6	2.04	0.343	0.336	4.356	0.367	0.711
F1	2.79	0.299	0.330	4.322	0.361	14.271
F2	2.53	0.350	0.350	4.311	0.375	3.026
F3	0.74	0.350	0.350	3.266	0.647	0.000
G1	1.12	0.350	0.350	4.688	0.298	0.000
H1	1.07	0.350	0.350	2.733	0.945	0.000
H2	1.40	0.350	0.350	3.222	0.664	0.000
J1	1.73	0.350	0.350	3.250	0.652	0.000
J2	3.64	0.349	0.348	3.623	0.495	0.281
K1	1.54	0.112	0.255	4.382	0.365	21.060
K2	2.37	0.341	0.346	4.471	0.348	1.018
K3	1.64	0.350	0.350	3.421	0.582	0.000
L1	3.55	0.271	0.318	4.357	0.352	16.317
L2	7.33	0.350	0.350	4.202	0.399	0.000
L3	5.85	0.341	0.328	3.980	0.419	1.050
M1	5.59	0.350	0.350	4.306	0.347	17.122
M2	7.53	0.350	0.350	3.952	0.446	0.287
N1	1.52	0.350	0.350	4.075	0.419	8.380
N2	3.33	0.350	0.350	4.117	0.405	2.670
O1	3.09	0.350	0.350	4.189	0.393	2.105
P1	2.44	0.350	0.350	4.193	0.396	6.712
Q1	2.23	0.350	0.350	4.203	0.390	12.313
R1	0.28	0.350	0.350	4.358	0.374	8.387
S1	1.41	0.350	0.350	4.484	0.352	6.918
T1	3.04	0.350	0.350	4.644	0.299	11.497

Subbasin	Area [mi^2]	IA	DTHETA	PSIF	XKSAT	RTIMP
U1	1.37	0.350	0.370	5.258	0.233	6.449
V1	0.66	0.350	0.350	3.972	0.405	13.864
W1	3.89	0.350	0.350	3.976	0.410	15.199
W2	2.85	0.350	0.350	4.081	0.402	3.547
X1	0.79	0.350	0.350	3.955	0.417	12.555
Y1	0.58	0.350	0.350	4.044	0.415	10.947
Z1	0.66	0.321	0.292	4.004	0.430	2.909

Table D.2-4

Summary of Green & Ampt Parameters for Buckeye Area 3 Future Conditions

Subbasin	Area [mi^2]	IA	DTHETA	PSIF	XKSAT	RTIMP
A1	1.11	0.250	0.250	4.058	0.472	39.677
A2	1.66	0.250	0.250	4.547	0.420	30.000
A3	3.73	0.250	0.250	4.230	0.491	30.000
B1	0.56	0.250	0.260	3.733	0.484	36.272
B2	0.49	0.250	0.260	3.617	0.542	30.000
B3	1.99	0.250	0.250	4.284	0.468	30.000
C1	3.59	0.181	0.262	3.933	0.412	31.606
C2	1.92	0.250	0.250	3.920	0.482	30.000
C3	3.79	0.250	0.250	4.357	0.444	30.000
C4	1.40	0.250	0.250	4.737	0.345	30.000
E1	0.96	0.250	0.250	5.377	0.273	30.690
E2	1.98	0.250	0.250	5.043	0.306	30.000
E3	2.49	0.250	0.250	4.358	0.377	30.000
E4	3.04	0.250	0.250	3.816	0.441	30.000
E5	1.31	0.250	0.250	3.940	0.473	30.000
E6	2.04	0.289	0.289	4.356	0.392	18.293
F1	2.79	0.240	0.270	4.322	0.384	32.052
F2	2.53	0.250	0.250	4.311	0.436	33.026
F3	0.74	0.250	0.270	3.266	0.656	30.000
G1	1.12	0.297	0.297	4.688	0.326	15.814
H1	1.07	0.250	0.290	2.733	0.945	30.000
H2	1.40	0.250	0.270	3.222	0.664	30.000
J1	1.73	0.250	0.270	3.250	0.657	30.000
J2	3.64	0.250	0.260	3.623	0.499	30.000
K1	1.54	0.112	0.255	4.382	0.365	21.060
K2	2.37	0.272	0.277	4.471	0.382	21.860
K3	1.64	0.250	0.270	3.421	0.600	30.000
L1	3.55	0.271	0.318	4.357	0.352	16.317
L2	7.33	0.240	0.257	4.202	0.462	33.542
L3	5.85	0.205	0.257	3.980	0.449	45.272
M1	5.59	0.349	0.349	4.306	0.348	17.433
M2	7.53	0.205	0.255	3.952	0.547	45.552
N1	1.52	0.289	0.289	4.075	0.469	26.648
N2	3.33	0.268	0.268	4.117	0.446	27.305
O1	3.09	0.252	0.252	4.189	0.443	31.440
P1	2.44	0.312	0.312	4.193	0.424	18.000
Q1	2.23	0.338	0.340	4.203	0.396	15.896
R1	0.28	0.181	0.282	4.358	0.474	62.581
S1	1.41	0.287	0.312	4.484	0.393	26.695
T1	3.04	0.327	0.327	4.644	0.315	18.464

Subbasin	Area [mi^2]	IA	DTHETA	PSIF	XKSAT	RTIMP
U1	1.37	0.317	0.331	5.258	0.251	16.201
V1	0.66	0.334	0.334	3.972	0.405	18.810
W1	3.89	0.350	0.350	3.976	0.410	15.199
W2	2.85	0.320	0.320	4.081	0.407	12.365
X1	0.79	0.350	0.350	3.955	0.417	12.555
Y1	0.58	0.350	0.350	4.044	0.415	10.947
Z1	0.66	0.168	0.277	4.004	0.499	58.182

Table D.2-5

Summary of Lag Time Estimation for Buckeye Area 3 Sub-basins

Subbasin	Area [mi^2]	L [mi]	L_{ca} [mi]	S [ft/mi]	S-Graph Type	K_n	Lag [min]
A1	1.11	2.20	0.86	322.08	Phx Mnt	0.050	30
A2	1.66	3.34	1.28	94.61	Des Rnglnd	0.055	57
A3	3.73	3.92	1.89	58.21	Des Rnglnd	0.045	64
B1	0.56	2.52	1.19	227.45	Phx Mnt	0.050	38
B2	0.49	2.21	0.98	84.60	Des Rnglnd	0.055	45
B3	1.99	3.72	1.65	63.31	Des Rnglnd	0.045	59
C1	3.59	3.99	2.01	387.22	Phx Mnt	0.050	51
C2	1.92	3.69	1.53	105.29	Des Rnglnd	0.055	63
C3	3.79	4.44	2.51	56.33	Des Rnglnd	0.050	84
C4	1.40	2.18	0.88	44.31	Des Rnglnd	0.045	40
E1	0.96	2.51	1.13	178.34	Phx Mnt	0.055	43
E2	1.98	2.92	0.80	96.39	Des Rnglnd	0.055	45
E3	2.49	3.87	1.99	49.65	Des Rnglnd	0.050	75
E4	3.04	5.36	2.81	38.26	Des Rnglnd	0.050	102
E5	1.31	3.01	1.41	28.56	Des Rnglnd	0.050	66
E6	2.04	3.04	1.06	34.17	Des Rnglnd	0.050	57
F1	2.79	4.07	1.63	441.98	Phx Mnt	0.050	46
F2	2.53	3.88	1.76	161.60	Phx Mnt	0.050	57
F3	0.74	2.71	1.26	81.34	Des Rnglnd	0.055	54
G1	1.12	2.38	1.08	33.76	Des Rnglnd	0.045	47
H1	1.07	2.41	1.09	64.59	Des Rnglnd	0.055	51
H2	1.40	4.41	2.60	44.71	Des Rnglnd	0.050	89
J1	1.73	4.09	1.78	70.82	Des Rnglnd	0.055	75
J2	3.64	5.18	2.51	45.01	Des Rnglnd	0.050	93
K1	1.54	3.23	1.84	681.00	Phx Mnt	0.050	40
K2	2.37	7.30	4.04	108.84	Des Rnglnd	0.055	119
K3	1.64	4.92	2.65	44.16	Des Rnglnd	0.050	94
L1	3.55	4.93	2.29	308.27	Phx Mnt	0.050	61
L2	7.33	5.21	2.40	82.13	Des Rnglnd	0.055	90

Subbasin	Area [mi^2]	L [mi]	L_{ca} [mi]	S [ft/mi]	S-Graph Type	K_n	Lag [min]
L3	5.85	4.75	2.00	44.27	Des Rnglnd	0.055	91
M1	5.59	4.99	1.81	302.46	Phx Mnt	0.050	56
M2	7.53	7.60	3.07	88.36	Des Rnglnd	0.050	103
N1	1.52	2.79	1.44	340.38	Phx Mnt	0.050	40
N2	3.33	5.64	3.27	170.14	Phx Mnt	0.055	91
O1	3.09	5.20	2.79	64.07	Phx Mnt	0.055	100
P1	2.44	6.39	2.62	200.13	Phx Mnt	0.055	85
Q1	2.23	5.23	2.81	228.56	Phx Mnt	0.055	78
R1	0.28	1.01	0.46	213.31	Phx Mnt	0.050	19
S1	1.41	2.32	0.52	143.99	Phx Mnt	0.050	29
T1	3.04	4.34	1.58	274.89	Phx Mnt	0.050	51
U1	1.37	2.37	1.03	326.50	Phx Mnt	0.050	33
V1	0.66	2.86	1.75	344.82	Phx Mnt	0.050	43
W1	3.89	3.46	1.78	486.28	Phx Mnt	0.050	44
W2	2.85	3.25	1.19	178.54	Phx Mnt	0.050	44
X1	0.79	2.60	1.44	288.89	Phx Mnt	0.050	40
Y1	0.58	1.37	0.53	771.84	Phx Mnt	0.050	17
Z1	0.66	1.70	0.66	47.00	Des Rnglnd	0.055	39

D.3 Hydrograph Routing Data

Table D.3-1

Summary of Routing Channel Parameters – Length, Slope and Manning's n

Routing Channel	L [ft]	S [ft/ft]	n-left	n-channel	n-right
A1-A2	15,153	0.0134	0.045	0.045	0.045
A2-A3	18,190	0.0116	0.050	0.043	0.050
B1-B2	11,334	0.0137	0.055	0.042	0.055
B2-B3	19,622	0.0121	0.050	0.025	0.050
C1-C2	18,437	0.0176	0.050	0.044	0.050
C2-C3	23,418	0.0107	0.046	0.050	0.046
E1-E2	11,365	0.0160	0.050	0.047	0.046
E2-E3	19,626	0.0088	0.052	0.046	0.052
E3-E4	13,855	0.0048	0.056	0.049	0.056
F3-E4	27,656	0.0072	0.050	0.044	0.050
E4H2-E5	15,206	0.0051	0.060	0.050	0.060
E5K3-E6	5,083	0.0037	0.063	0.051	0.063
F1-F2	8,635	0.0120	0.053	0.045	0.055
F2-F3	7,059	0.0100	0.050	0.047	0.048
H1-H2	21,738	0.0082	0.053	0.050	0.053
J1-J2	24,090	0.0086	0.055	0.040	0.055

Routing Channel	L [ft]	S [ft/ft]	n-left	n-channel	n-right
K1-K2	30,021	0.0150	0.048	0.050	0.048
K2-K3	25,950	0.0084	0.050	0.048	0.050
J2-K3	4,072	0.0059	0.050	0.048	0.050
L1-L2	28,949	0.0147	0.046	0.045	0.046
L2-L3	20,567	0.0084	0.050	0.040	0.060
M1-M2	38,719	0.0133	0.040	0.040	0.045
N1-N2	26,511	0.0115	0.05	0.047	0.05
W1-W2	11,249	0.0189	0.055	0.050	0.060

Table D.3-2

Summary of Routing Channel Parameters – Representative Cross Sections

	A2		A3		B2		B3		C2		C3		E2		E3	
	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
1	0	27.0	0	20.0	0	20.0	0	13.5	0	21.0	0	26.0	0	24.0	0	22.0
2	75	23.0	20	19.0	75	17.0	120	10.0	35	19.0	60	25.0	65	21.0	133	17.0
3	125	19.0	30	18.0	140	14.0	278	11.6	90	16.0	125	23.0	140	17.0	173	14.0
4	185	10.0	165	10.0	167	10.0	285	10.1	138	10.0	140	10.0	170	10.0	213	10.0
5	190	10.0	186	10.0	193	10.0	301.5	10.1	159	10.0	172	10.0	185	10.0	223	10.0
6	220	20.0	346	20.0	226	13.0	312.5	11.5	209	15.0	232	16.0	200	14.0	263	15.0
7	280	26.0	361	21.0	296	17.0	446.5	10.8	264	20.0	292	20.0	270	18.0	303	18.0
8	340	32.0	381	22.0	371	22.0	529.5	12.6	309	24.0	352	23.0	340	22.0	403	22.0

	E4A		E4B		E5		E6		F2		F3		H2		J2	
	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
1	0	21.0	0	20.0	0	27.0	0	25.0	0	27.0	0	20.0	0	17.0	0	17.0
2	570	14.0	570	14.0	1010	16.0	850	13.0	60	26.0	350	15.0	330	14.0	161	12.5
3	650	13.0	650	13.0	1080	15.0	980	13.0	120	25.0	425	13.0	430	12.0	247	12.2
4	690	10.0	690	10.0	1090	10.0	990	10.0	146	10.0	455	10.0	465	10.0	254	10.8
5	705	10.0	705	10.0	1110	10.0	1030	10.0	178	10.0	472	10.0	495	10.0	271	10.0
6	735	12.5	735	12.5	1120	14.0	1040	12.0	208	24.0	499	12.0	530	12.0	279	12.1
7	810	14.5	810	14.5	1190	16.0	1230	12.0	268	26.0	569	14.0	630	14.0	393	12.9
8	1200	21.0	1200	20.0	2200	27.0	2020	25.0	328	28.0	900	20.0	960	17.0	660	17.0

	K2		K3A		K3B		L2		L3		M2		N2		W2	
	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
1	0	22.0	0	20.0	0	20.0	0	19.0	0	18	0	15.5	0	17.0	0	17.5
2	20	21.0	390	14.5	390	14.5	80	17.0	1093	11.8	961	12.0	280	13.0	350	11.5
3	55	20.0	490	11.5	490	11.5	165	14.0	1210	11.7	1028	11.8	350	11.5	372	11.6
4	85	10.0	502	10.0	502	10.0	200	10.0	1215	10	1051	10.3	358	10.0	384	10.0
5	235	10.0	520	10.0	520	10.0	215	10.0	1225	10	1136	10.0	367	10.0	472	11.2
6	259	20.0	532	11.5	532	11.5	255	14.0	1232	11.7	1153	11.0	381	11.5	489	13.4
7	289	21.0	632	14.5	632	14.5	340	18.0	1372	11.8	1193	11.2	451	13.0	525	13.8
8	309	22.0	1022	20.0	1022	20.0	420	22.0	2000	18	1900	15.5	730	17.0	1005	17.5

Table D.3-3

Summary of Routing Channel Parameters – NSTEPS Used in HEC-1 Models (Existing Conditions)

Routing Channel	10-yr 6-hr	10-yr 24-hr	100-yr 6-hr	100-yr 24-hr	200-yr 24-hr	500-yr 24-hr	PMP 6-hr	PMP 72-hr
A1-A2	9	9	7	7	7	7	6	6
A2-A3	13	12	10	9	9	9	8	8
B1-B2	8	9	7	8	8	7	6	6
B2-B3	22	22	23	23	23	21	16	16
C1-C2	8	7	6	6	6	5	5	5
C2-C3	13	11	9	10	10	8	8	8
E1-E2	7	7	5	5	5	5	5	5
E2-E3	13	13	9	9	8	8	8	8
E3-E4	13	13	14	13	13	13	9	11
F3-E4	19	19	19	19	19	19	16	18
E4H2-E5	17	17	16	14	13	13	9	11
E5K3-E6	7	7	6	5	5	5	4	4
F1-F2	4	4	3	3	3	3	3	2
F2-F3	5	5	5	4	4	4	4	5
H1-H2	31	31	19	19	17	17	16	16
J1-J2	17	17	28	26	25	25	16	19
K1-K2	26	26	17	17	15	15	9	13
K2-K3	22	22	19	18	19	19	19	19
J2-K3	5	5	4	4	4	4	3	3
L1-L2	14	14	11	10	10	10	10	10
L2-L3	31	23	19	17	17	17	11	11
M1-M2	26	25	26	26	26	26	20	20
N1-N2	23	23	21	21	21	21	17	17
W1-W2	7	6	6	6	6	6	6	6

Table D.3-4

Summary of Routing Channel Parameters – NSTEPS Used in HEC-1 Models (Future Conditions)

Routing Channel	10-yr 6-hr	10-yr 24-hr	100-yr 6-hr	100-yr 24-hr	200-yr 24-hr	500-yr 24-hr	PMP 6-hr	PMP 72-hr
E1-E2	40	73	10	10	9	7	5	5
E2-E3	38	39	15	14	13	12	8	8
E3-E4	1	48	14	14	14	14	9	11
F3-E4	1	37	21	21	21	21	16	17
E4H2-E5	1	33	17	16	15	15	9	11

Routing Channel	10-yr 6-hr	10-yr 24-hr	100-yr 6-hr	100-yr 24-hr	200-yr 24-hr	500-yr 24-hr	PMP 6-hr	PMP 72-hr
E5K3-E6	11	12	6	6	5	5	3	4
F1-F2	22	28	8	6	5	4	3	2
F2-F3	1	16	6	6	5	5	4	5
H1-H2	55	151	31	32	27	23	17	17
J1-J2	1	1	38	37	35	30	17	19
K1-K2	23	25	17	18	17	16	11	13
K2-K3	25	26	21	20	20	20	18	20
J2-K3	1	1	6	6	6	6	4	4
L1-L2	14	14	11	11	10	10	10	10
L2-L3	35	34	22	22	20	20	20	11
M1-M2	56	56	30	30	29	29	28	21
N1-N2	48	47	26	26	24	24	23	17
W1-W2	7	6	6	6	6	6	6	6

D.4 Reservoir Routing Data

N/A

D.5 Flow Splits and Diversion Data

Table D.5-1

Increase of Peak and Runoff Volume in 100-Year 24-Hour Storm Event in Future Conditions

Station	Area [mi2]	100-YR 24-HR (existing)		100-YR 24-HR (future w/out retention)		% Increase	
		Peak [cfs]	Volume [ac-ft]	Peak [cfs]	Volume [ac-ft]	Peak	Volume
E	31.24	6068	1637	8616	2923	41.99%	78.56%
E1	0.96	790	63	904	107	14.43%	69.84%
E1-E2	0.96	679	63	802	107	18.11%	69.84%
E2	1.98	1548	123	1808	216	16.80%	75.61%
E2-E3	2.93	1684	184	2124	321	26.13%	74.46%
E3	2.49	1129	143	1443	265	27.81%	85.31%
E3-E4	5.42	2191	325	2950	582	34.64%	79.08%
E3R	2.93	2008	184	2422	321	20.62%	74.46%
E4	3.04	966	164	1332	319	37.89%	94.51%
E4H2-E5	16.98	4752	944	6611	1713	39.12%	81.46%
E4RA	5.42	2619	325	3436	582	31.20%	79.08%
E4RB	6.05	3398	406	4075	646	19.92%	59.11%

Station	Area [mi2]	100-YR 24-HR (existing)		100-YR 24-HR (future w/out retention)		% Increase	
		Peak [cfs]	Volume [ac-ft]	Peak [cfs]	Volume [ac-ft]	Peak	Volume
E5	1.31	645	74	819	136	26.98%	83.78%
E5K2-E6	29.2	6068	1531	8629	2773	42.21%	81.12%
E5R	16.98	5270	944	7116	1713	35.03%	81.46%
E6	2.04	1194	118	1356	176	13.57%	49.15%
E6R	29.2	6229	1531	8795	2773	41.19%	81.12%
ECP	31.24	6027	1628	8613	2922	42.91%	79.48%
F1	2.79	2147	223	2376	303	10.67%	35.87%
F1-F2	2.79	2005	223	2235	303	11.47%	35.87%
F2	2.53	1506	153	1828	275	21.38%	79.74%
F2-F3	5.31	3110	373	3667	574	17.91%	53.89%
F3	0.74	374	34	512	74	36.90%	117.65%
F3-E4	3.05	2583	406	3202	646	23.96%	59.11%
F3R	5.31	3484	373	4034	574	15.79%	53.89%
G1	1.12	820	68	899	96	9.63%	41.18%
GCP	1.12	820	68	899	96	9.63%	41.18%
H1	1.07	472	40	694	99	47.03%	147.50%
H1-H2	1.07	305	40	537	99	76.07%	147.50%
H2	1.4	432	64	638	139	47.69%	117.19%
J1	1.73	633	79	909	172	43.60%	117.72%
J1-J2	1.73	488	79	741	172	51.84%	117.72%
J2	3.64	1213	189	1679	374	38.42%	97.88%
J2-K3	5.37	1186	266	1805	543	52.19%	104.14%
K1	1.54	1432	146	1432	146	0.00%	0.00%
K1-K2	1.54	813	146	835	146	2.71%	0.00%
K2	2.37	694	139	852	219	22.77%	57.55%
K2-K3	3.91	1216	283	1390	363	14.31%	28.27%
K3	1.64	507	79	724	165	42.80%	108.86%
K3RA	3.91	1456	283	1640	363	12.64%	28.27%
K3RB	5.37	1207	266	1837	543	52.20%	104.14%
L	16.73	3781	976	5288	1770	39.86%	81.35%
L1	3.55	2267	298	2267	298	0.00%	0.00%
L1-L2	3.55	1803	298	1832	298	1.61%	0.00%
L2	7.33	2641	397	3556	791	34.65%	99.24%
L2-L3	10.88	3494	685	4412	1078	26.27%	57.37%
L3	5.85	2129	330	3075	744	44.43%	125.45%
L3R	10.88	4184	685	5103	1078	21.96%	57.37%
LCP	16.73	3781	976	5288	1770	39.86%	81.35%
M	13.12	3327	841	4136	1382	24.32%	64.33%
M1	5.59	3717	467	3723	469	0.16%	0.43%
M1-M2	5.59	2554	467	2559	469	0.20%	0.43%
M2	7.53	2304	397	3470	941	50.61%	137.03%
MCP	13.12	3327	841	4136	1382	24.32%	64.33%

Station	Area [mi2]	100-YR 24-HR (existing)		100-YR 24-HR (future w/out retention)		% Increase	
		Peak [cfs]	Volume [ac-ft]	Peak [cfs]	Volume [ac-ft]	Peak	Volume
N	4.85	1364	297	1749	475	28.23%	59.93%
N1	1.52	1211	103	1346	148	11.15%	43.69%
N1-N2	1.52	757	103	880	148	16.25%	43.69%
N2	3.33	1331	196	1633	330	22.69%	68.37%
NCP	4.85	1364	297	1749	475	28.23%	59.93%
O1	3.09	1163	180	1476	328	26.91%	82.22%
OCP	3.09	1163	180	1476	328	26.91%	82.22%
P1	2.44	1073	160	1171	204	9.13%	27.50%
PCP	2.44	1073	160	1171	204	9.13%	27.50%
Q1	2.23	1111	167	1141	180	2.70%	7.78%
QCP	2.23	1111	167	1141	180	2.70%	7.78%
R1	0.28	387	20	465	44	20.16%	120.00%
RCP	0.28	387	20	465	44	20.16%	120.00%
S1	1.41	1449	95	1575	137	8.70%	44.21%
SCP	1.41	1449	95	1575	137	8.70%	44.21%
T1	3.04	2151	234	2236	267	3.95%	14.10%
TCP	3.04	2151	234	2236	267	3.95%	14.10%
U1	1.37	1365	100	1427	122	4.54%	22.00%
UCP	1.37	1365	100	1427	122	4.54%	22.00%
V1	0.66	526	52	545	57	3.61%	9.62%
VCP	0.66	526	52	545	57	3.61%	9.62%
W	6.74	3648	471	3,766	512	3.23%	8.70%
W1	3.89	2976	304	2,976	304	0.00%	0.00%
W1-W2	3.89	2573	304	2,573	304	0.00%	0.00%
W2	2.85	2042	171	2,180	212	6.76%	23.98%
WCP	6.74	3648	471	3,766	512	3.23%	8.70%
X1	0.79	645	59	645	59	0.00%	0.00%
XCP	0.79	645	59	645	59	0.00%	0.00%
Y1	0.58	837	42	835	42	-0.24%	0.00%
YCP	0.58	837	42	835	42	-0.24%	0.00%
Z1	0.66	558	41	724	99	29.75%	141.46%
ZCP	0.66	558	41	724	99	29.75%	141.46%

Table D.5-2

Comparison of Peak and Runoff Volume between Existing and Future Conditions in 100-Year 24-Hour Storm Event

Station	Area [mi2]	100-YR 24-HR (existing)		Future Retention Volume [ac-ft]	100-YR 24-HR (future w/ retention)		% Increase	
		Peak [cfs]	Volume [ac-ft]		Peak [cfs]	Volume [ac-ft]	Peak [cfs]	Volume [ac-ft]
E	31.24	6068	1637		5088	1492	-16.15%	-8.86%
E1	0.96	790	63	53	904	107	14.43%	69.84%
E1-E2	0.96	679	63		505	54	-25.63%	-14.29%
E2	1.98	1548	123	118	1808	216	16.80%	75.61%
E2-E3	2.93	1684	184		1005	150	-40.32%	-18.48%
E3	2.49	1129	143	152	1443	265	27.81%	85.31%
E3-E4	5.42	2191	325		1365	259	-37.70%	-20.31%
E3R	2.93	2008	184		1455	150	-27.54%	-18.48%
E4	3.04	966	164	190	1332	319	37.89%	94.51%
E4H2-E5	16.98	4752	944		3148	744	-33.75%	-21.19%
E4RA	5.42	2619	325		1633	259	-37.65%	-20.31%
E4RB	6.05	3398	406		2694	338	-20.72%	-16.75%
E5	1.31	645	74	79	819	136	26.98%	83.78%
E5K2-E6	29.2	6068	1531		5063	1415	-16.56%	-7.58%
E5R	16.98	5270	944		3517	744	-33.26%	-21.19%
E6	2.04	1194	118	100	1356	176	13.57%	49.15%
E6R	29.2	6229	1531		5183	1415	-16.79%	-7.58%
ECP	31.24	6027	1628		5088	1492	-15.58%	-8.35%
F1	2.79	2147	223	133	2376	303	10.67%	35.87%
F1-F2	2.79	2005	223		1743	170	-13.07%	-23.77%
F2	2.53	1506	153	130	1828	275	21.38%	79.74%
F2-F3	5.31	3110	373		2483	311	-20.16%	-16.62%
F3	0.74	374	34	45	512	74	36.90%	117.65%
F3-E4	3.05	2583	406		1898	338	-26.52%	-16.75%
F3R	5.31	3484	373		2978	311	-14.52%	-16.62%
G1	1.12	820	68	47	899	96	9.63%	41.18%
GCP	1.12	820	68		786	49	-4.15%	-27.94%
H1	1.07	472	40	60	694	99	47.03%	147.50%
H1-H2	1.07	305	40		189	39	-38.03%	-2.50%
H2	1.4	432	64	88	638	139	47.69%	117.19%
J1	1.73	633	79	102	909	172	43.60%	117.72%
J1-J2	1.73	488	79		377	70	-22.75%	-11.39%
J2	3.64	1213	189	224	1679	374	38.42%	97.88%

Station	Area [mi2]	100-YR 24-HR (existing)		Future Retention Volume [ac-ft]	100-YR 24-HR (future w/ retention)		% Increase	
		Peak [cfs]	Volume [ac-ft]		Peak [cfs]	Volume [ac-ft]	Peak [cfs]	Volume [ac-ft]
J2-K3	5.37	1186	266		1002	217	-15.51%	-18.42%
K1	1.54	1432	146	0	1432	146	0.00%	0.00%
K1-K2	1.54	813	146		822	146	1.11%	0.00%
K2	2.37	694	139	128	852	219	22.77%	57.55%
K2-K3	3.91	1216	283		1007	238	-17.19%	-15.90%
K3	1.64	507	79	101	724	165	42.80%	108.86%
K3RA	3.91	1456	283		1350	238	-7.28%	-15.90%
K3RB	5.37	1207	266		1174	217	-2.73%	-18.42%
L	16.73	3781	976		2884	838	-23.72%	-14.14%
L1	3.55	2267	298	0	2267	298	0.00%	0.00%
L1-L2	3.55	1803	298		1832	298	1.61%	0.00%
L2	7.33	2641	397	475	3556	791	34.65%	99.24%
L2-L3	10.88	3494	685		2583	603	-26.07%	-11.97%
L3	5.85	2129	330	457	3075	744	44.43%	125.45%
L3R	10.88	4184	685		3810	603	-8.94%	-11.97%
LCP	16.73	3781	976		2884	838	-23.72%	-14.14%
M	13.12	3327	841		2948	627	-11.39%	-25.45%
M1	5.59	3717	467	170	3723	469	0.16%	0.43%
M1-M2	5.59	2554	467		1806	299	-29.29%	-35.97%
M2	7.53	2304	397	585	3470	941	50.61%	137.03%
MCP	13.12	3327	841		2948	627	-11.39%	-25.45%
N	4.85	1364	297		1276	260	-6.45%	-12.46%
N1	1.52	1211	103	65	1346	148	11.15%	43.69%
N1-N2	1.52	757	103		533	83	-29.59%	-19.42%
N2	3.33	1331	196	150	1633	330	22.69%	68.37%
NCP	4.85	1364	297		1276	260	-6.45%	-12.46%
O1	3.09	1163	180	150	1476	328	26.91%	82.22%
OCP	3.09	1163	180		1096	178	-5.76%	-1.11%
P1	2.44	1073	160	85	1171	204	9.13%	27.50%
PCP	2.44	1073	160		1064	119	-0.84%	-25.63%
Q1	2.23	1111	167	72	1141	180	2.70%	7.78%
QCP	2.23	1111	167		1059	109	-4.68%	-34.73%
R1	0.28	387	20	24	465	44	20.16%	120.00%
RCP	0.28	387	20		296	20	-23.51%	0.00%
S1	1.41	1449	95	61	1572	136	8.49%	43.16%
SCP	1.41	1449	95		1339	75	-7.59%	-21.05%
T1	3.04	2151	234	103	2231	266	3.72%	13.68%
TCP	3.04	2151	234		2067	163	-3.91%	-30.34%
U1	1.37	1365	100	47	1424	121	4.32%	21.00%
UCP	1.37	1365	100		1309	74	-4.10%	-26.00%

Station	Area [mi2]	100-YR 24-HR (existing)		Future Retention Volume [ac-ft]	100-YR 24-HR (future w/ retention)		% Increase	
		Peak [cfs]	Volume [ac-ft]		Peak [cfs]	Volume [ac-ft]	Peak [cfs]	Volume [ac-ft]
V1	0.66	526	52	23	545	57	3.61%	9.62%
VCP	0.66	526	52		487	34	-7.41%	-34.62%
W	6.74	3648	471		3575	382	-2.00%	-18.90%
W1	3.89	2976	304	0	2976	304	0.00%	0.00%
W1-W2	3.89	2573	304		2573	304	0.00%	0.00%
W2	2.85	2042	171	130	2180	212	6.76%	23.98%
WCP	6.74	3648	471		3575	382	-2.00%	-18.90%
X1	0.79	645	59	0	645	59	0.00%	0.00%
XCP	0.79	645	59		645	59	0.00%	0.00%
Y1	0.58	837	42	0	835	42	-0.24%	0.00%
YCP	0.58	837	42		835	42	-0.24%	0.00%
Z1	0.66	558	41	58	724	99	29.75%	141.46%
ZCP	0.66	558	41		476	41	-14.70%	0.00%

D.6 Hydrologic Calculations

Table D.6-1

Comparison of Peak Discharge for 15-Minute and 30-Minute Intervals

	15 min Interval		30 min Interval			
	Peak [cfs]	Tp [hours]	Peak [cfs]	Tp [hours]	Peak % Diff	Tp % Diff
A	2,433	13.42	2,410	13.33	0.9%	0.7%
A1	1,104	12.33	963	12.25	12.8%	0.6%
A1-A2	846	12.83	804	12.75	5.0%	0.6%
A2	997	12.75	978	12.67	1.9%	0.6%
A2-A3	1,495	13.5	1,463	13.42	2.1%	0.6%
A3	1,855	12.92	1,823	12.75	1.7%	1.3%
A3R	1,808	12.83	1,742	12.75	3.7%	0.6%
ACP	2,433	13.42	2,410	13.33	0.9%	0.7%
B	1,121	12.75	1,102	12.67	1.7%	0.6%
B1	448	12.42	398	12.33	11.2%	0.7%
B1-B2	363	13	349	12.92	3.9%	0.6%
B2	338	12.58	322	12.5	4.7%	0.6%
B2-B3	411	14.67	405	14.58	1.5%	0.6%
B3	1,125	12.75	1,107	12.67	1.6%	0.6%
B3R	566	12.83	551	12.75	2.7%	0.6%
BCP	1,121	12.75	1,102	12.67	1.7%	0.6%
C	3,793	13.67	3,728	13.58	1.7%	0.7%
C1	2,633	12.67	2,417	12.5	8.2%	1.3%

	15 min Interval		30 min Interval			
	Peak [cfs]	Tp [hours]	Peak [cfs]	Tp [hours]	Peak % Diff	Tp % Diff
C1-C2	2,244	13.08	2,205	13	1.7%	0.6%
C2	998	12.83	979	12.75	1.9%	0.6%
C2-C3	2,718	13.75	2,655	13.67	2.3%	0.6%
C3	1,506	13.17	1,487	13.08	1.3%	0.7%
C3R	3,074	13	3,016	13	1.9%	0.0%
C4	1,115	12.58	1,071	12.5	3.9%	0.6%
C4R	3,709	13.67	3,636	13.58	2.0%	0.7%
CCP	3,793	13.67	3,728	13.58	1.7%	0.7%

Table D.6-2

Comparison of Phoenix Valley and Desert/Rangeland S-Graphs

	Desert Rangeland		Phoenix Valley			
	Peak [cfs]	T_p [hr]	Peak [cfs]	T_p [hr]	Peak % Diff	Tp % Diff
A	2,461	13.42	2782	13.42	13.0%	0.0%
A1	1,104	12.33	1104	12.33	0.0%	0.0%
A1-A2	846	12.83	892	12.83	5.4%	0.0%
A2	997	12.75	1178	12.83	18.2%	0.6%
A2-A3	1,522	13.50	1697	13.5	11.5%	0.0%
A3	1,855	12.92	2227	13	20.1%	0.6%
A3R	1,808	12.83	2051	12.83	13.4%	0.0%
B	1,121	12.75	1319	12.92	17.7%	1.3%
B1	448	12.42	448	12.42	0.0%	0.0%
B1-B2	363	13.00	371	13	2.2%	0.0%
B2	338	12.58	389	12.67	15.1%	0.7%
B2-B3	407	14.67	423	14.58	3.9%	-0.6%
B3	1,125	12.75	1324	12.83	17.7%	0.6%
B3R	566	12.83	609	12.83	7.6%	0.0%
C	3,756	13.67	4098	13.67	9.1%	0.0%
C1	2,633	12.67	2633	12.67	0.0%	0.0%
C1-C2	2,244	13.08	2288	13.08	2.0%	0.0%
C2	998	12.83	1197	12.92	19.9%	0.7%
C2-C3	2,684	13.75	2888	13.75	7.6%	0.0%
C3	1,506	13.17	1852	13.25	23.0%	0.6%
C3R	3,074	13.00	3322	13	8.1%	0.0%
C4	1,115	12.58	1275	12.67	14.3%	0.7%

The detailed output for PMP event HEC-1 models can be found on the attached CD. The following lists the sub-title for each event.

D.6.1 HEC-1 Output File PMP 6-Hour

D.6.1.a HEC-1 Output File for PMP 6-Hour (Existing Conditions)

D.6.1.b HEC-1 Output File for PMP 6-Hour (Future Conditions)

D.6.2 HEC-1 Output File PMP 72-Hour

D.6.2.a HEC-1 Output File for PMP 72-Hour (Existing Conditions)

D.6.2.b HEC-1 Output File for PMP 72-Hour (Future Conditions)

E. Hydraulic Analysis Supporting Documentations

E.1 Roughness Coefficient Estimation

See Section 5.3.1.

E.2 Cross Section Plots

See the avi clips in the attached CD.

E.3 Expansion and Contraction Coefficients

See Section 5.3.2.

E.4 Analysis of Structures

See Section 5.9.1.

E.5 Hydraulic Calculations

The detailed output for PMF events unsteady hydraulic models and level pool routing models can be found on the attached CD. The following lists the sub-title for each event.

E.5.1 FRS#1 PMF Unsteady Model

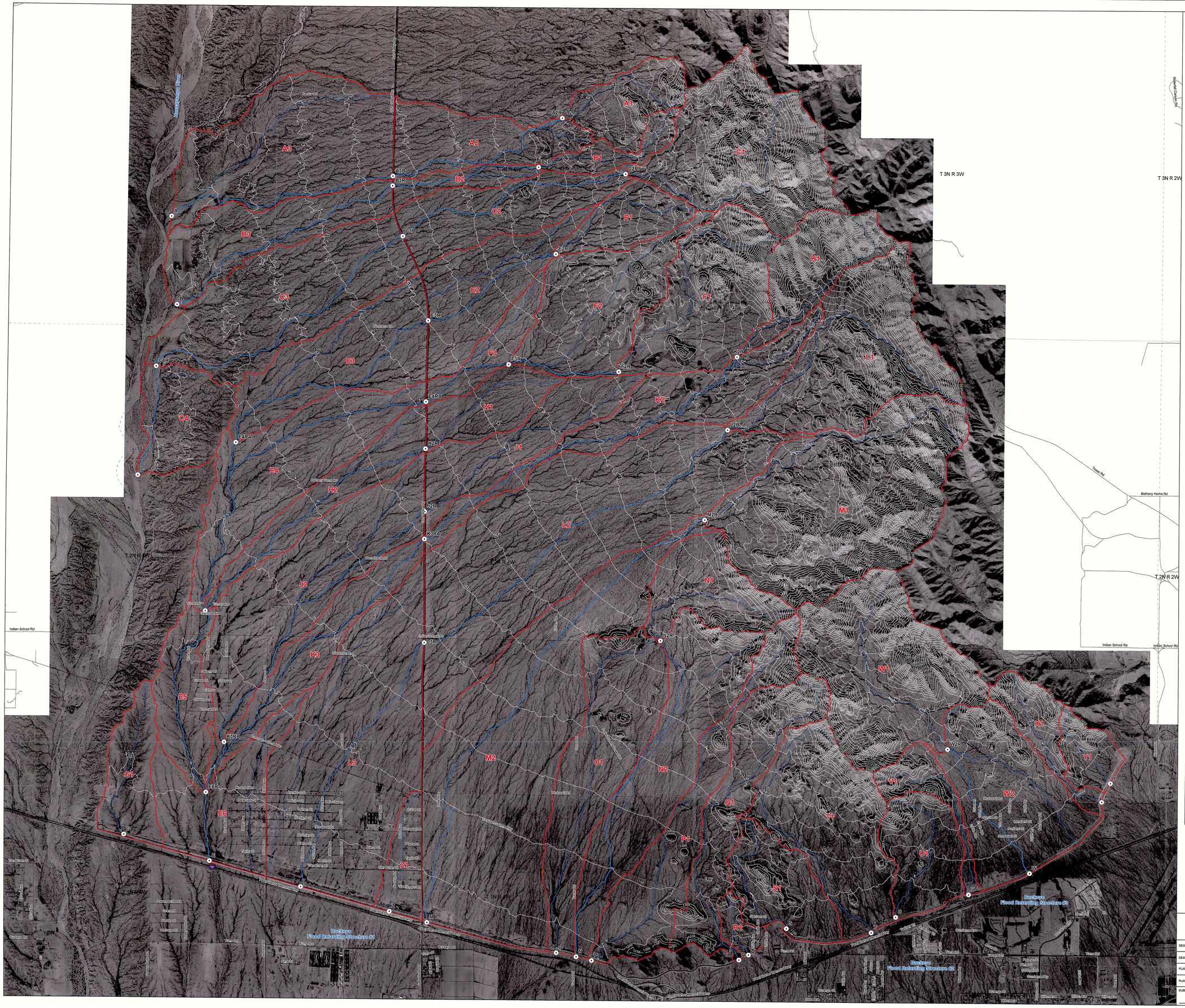
E.5.2 FRS#2 PMF Unsteady Model

E.5.3 FRS#3 PMF Unsteady Model

E.5.4 FRS#1 PMF Level Pool Routing

E.5.5 FRS#2 PMF Level Pool Routing

E.5.6 FRS#3 PMF Level Pool Routing



Flood Control District of Maricopa County

BUCKEYE/SUN VALLEY
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FCD CONTRACT NO. 2002C027

Figure 4-1
Area 3 Watershed Workmap

LEGEND

- Concentration/Combining Point
- E1 Sub-Basin Boundary
- Road
- Township - Range
- Contour
- Lag Time Path
- Routing Channel
- Culvert

NOTES

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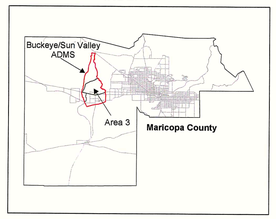
INDEX MAP



0 2,000 4,000 6,000 Feet

1 inch equals 2,000 feet
contour interval equals 50 feet

Horizontal Coordinate System:
NAD 1983 State Plane Arizona Central
Vertical Coordinate System:
NAVD 1988

N

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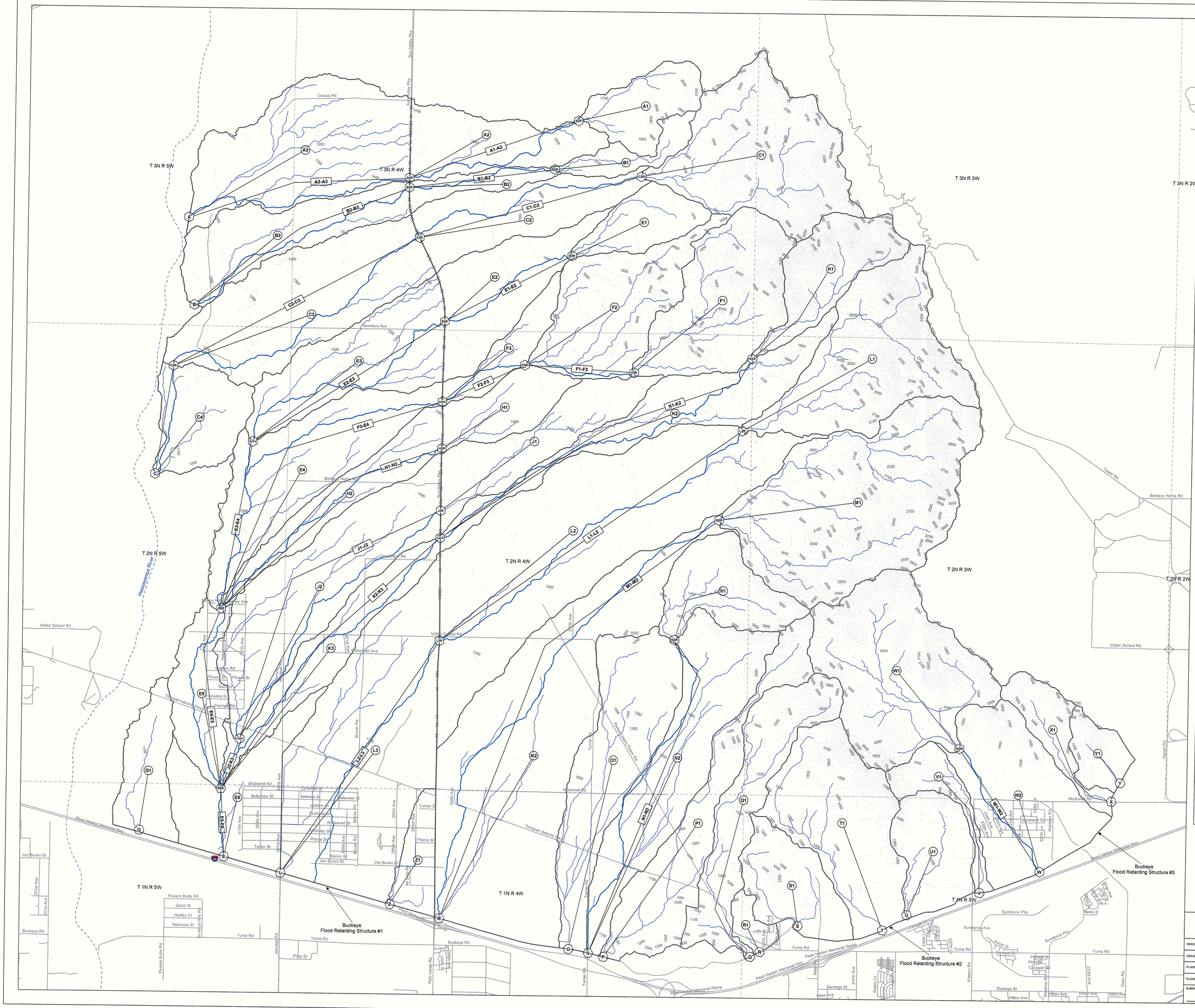
PLANS CHK.			APPROVED BY: DATE
SUBMITTED BY: DATE			CHIEF ENGINEER AND GENERAL MANAGER
			SHEET 1 OF 1



BUCKEYE/SUN VALLEY
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FCD CONTRACT NO. 2002C027

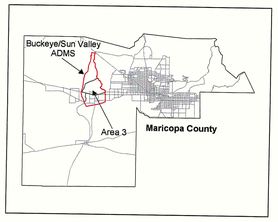
Figure 4-1A
Area 3 Existing Conditions Schematic Map

LEGEND

- (E1) Sub-Basin ID
- (E) Concentration/Combining Point
- E1-E2 Routing Channel ID
- Sub-Basin Boundary
- Routing Channel
- Drainage Path
- Culvert
- Road
- Contour (50ft)
- Township - Range

NOTES

INDEX MAP



0 2,000 4,000 6,000 Feet
1 inch equals 2,000 feet

N

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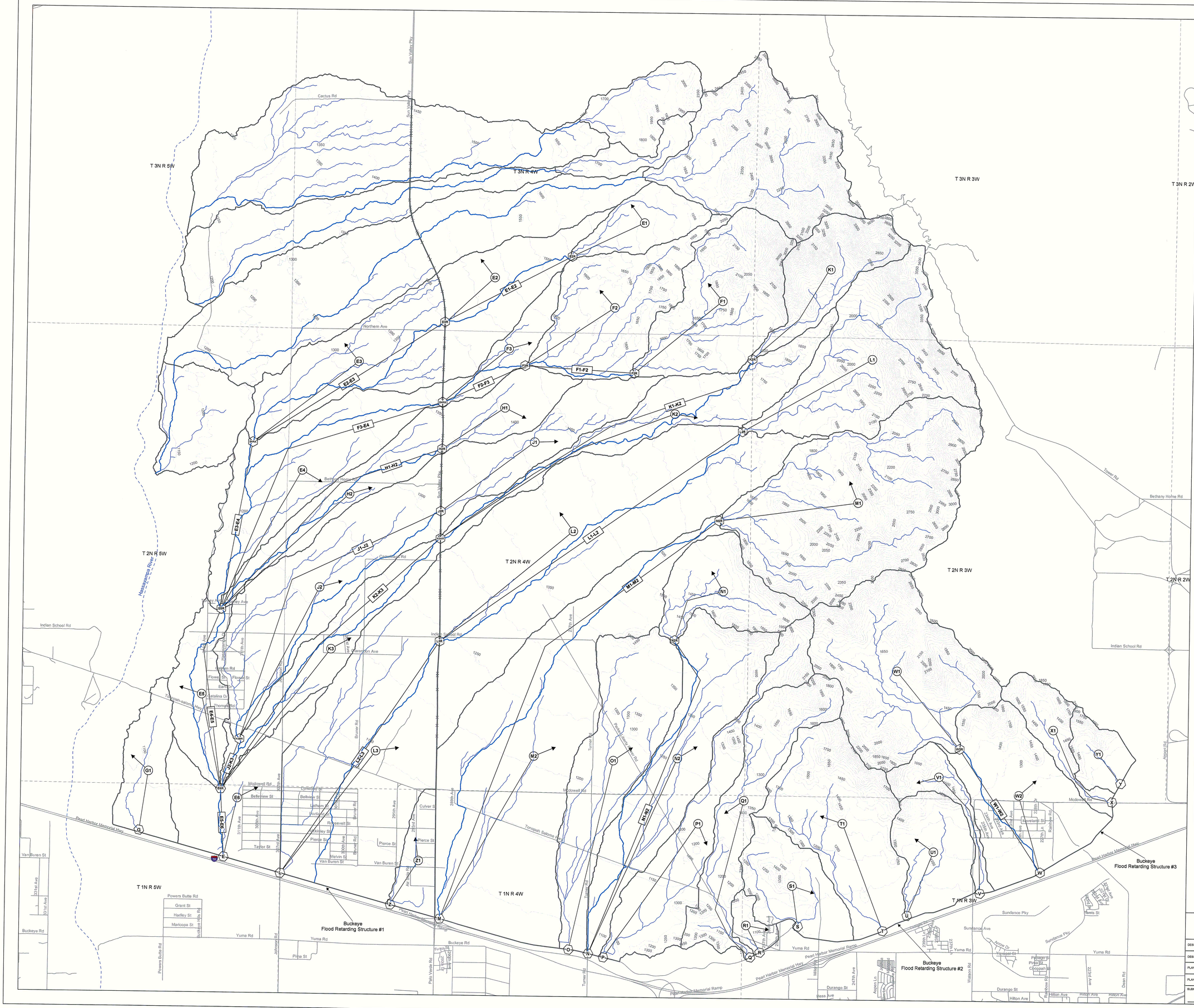

Flood Control District of Maricopa County

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY
FCD CONTRACT NO. 2002C027

Figure 4-1B
Area 3 Future Conditions Schematic Map

LEGEND

- E1 — Sub-Basin ID
- E — Concentration/Combining Point
- E1-E2 — Routing Channel ID
- Diversion to Retention Basin
- Sub-Basin Boundary
- Routing Channel
- Drainage Path
- Culvert
- Road
- Contour (50ft)
- Township - Range

NOTES

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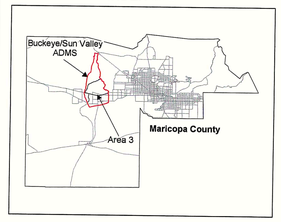
INDEX MAP



0 2,000 4,000 6,000 Feet

1 inch equals 2,000 feet

N

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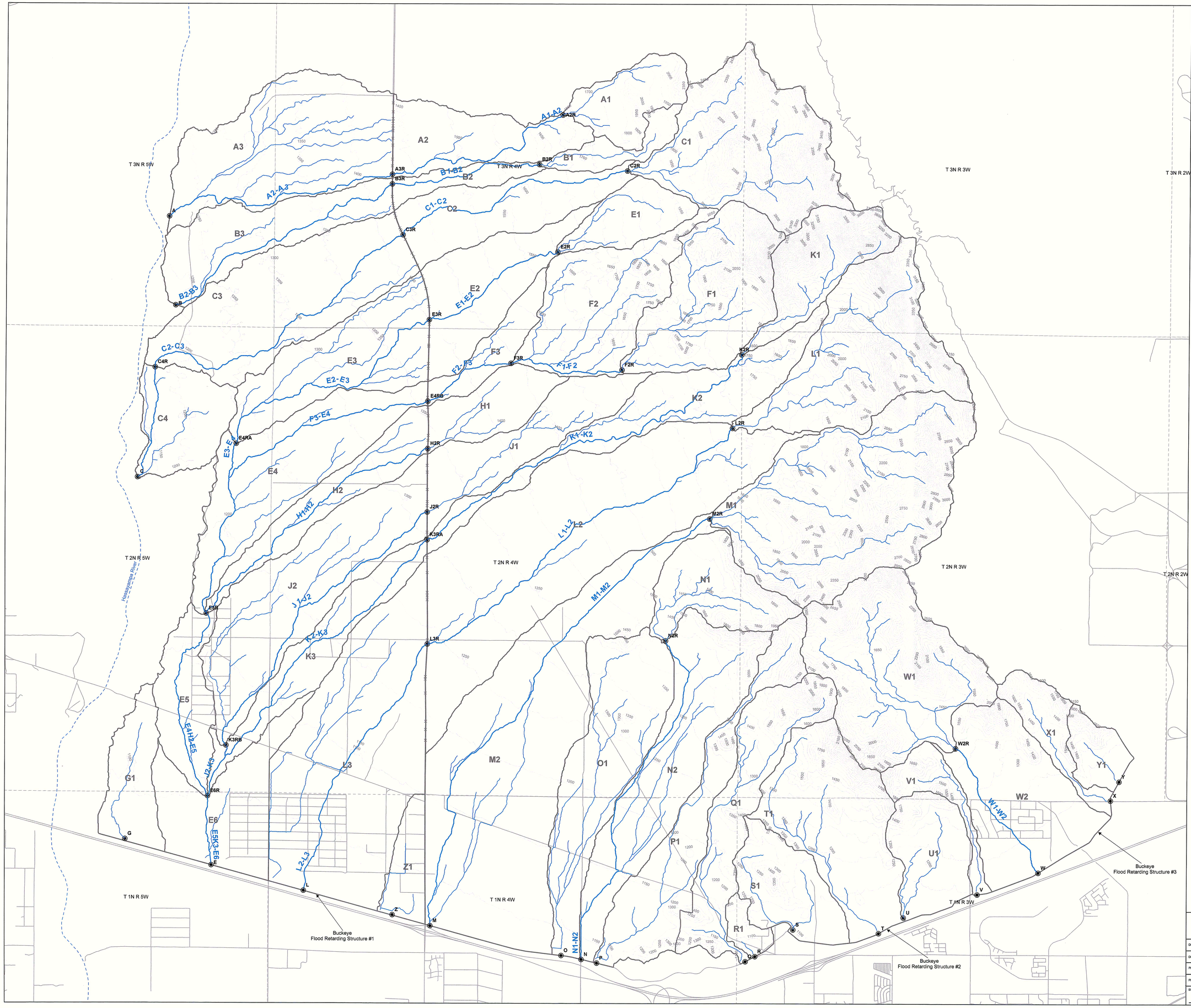

Flood Control District of Maricopa County

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY
FCD CONTRACT NO. 2002C027

Figure 4-2
Area 3 Routing Map

LEGEND

- E1 — Sub-Basin Boundary
- E — Concentration Point
- E1-E2 — Routing Channel
- Drainage Path
- Culvert
- Road
- Contour (50ft)
- Township - Range

NOTES

Please refer to Volume V-A Area 3 Hydrology Report Appendix E for cross section data, "n" value selection, culvert dimensions, slopes, and all other hydraulic data.

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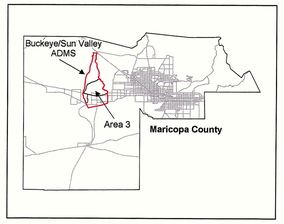
INDEX MAP



0 2,000 4,000 6,000 Feet

1 inch equals 2,000 feet

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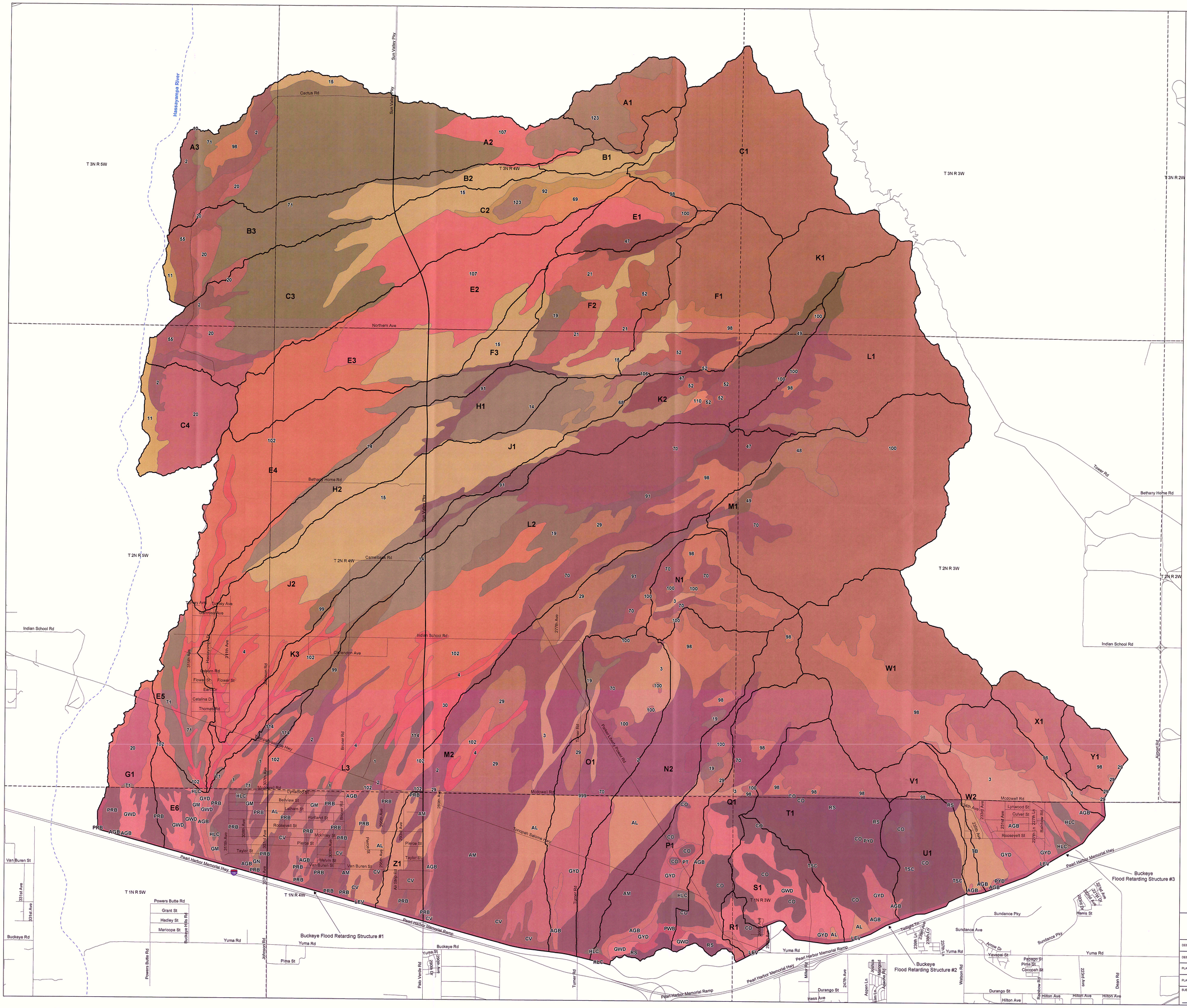

Flood Control District of Maricopa County

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Figure 4-3
Area 3 Soil Map

LEGEND

- E1 Sub-Basin Boundary
- Aba Soil Type
- Township-Range
- Roads

NOTES

Table D.2-2 lists the soil characteristics for each ID number.

Map shows soil types from two different surveys conducted by the USDA SCS. A horizontal line can be found in the southern part of area 3 where the two surveys join. The southern portion of the map includes the Soil Survey of Maricopa County, Arizona, Central Part (1972). The northern portion includes the Soil Survey of Aguila-Carafree Area, Parts of Maricopa and Pinal Counties, Arizona (1982). Hydrologic soil parameters from the two surveys were averaged.

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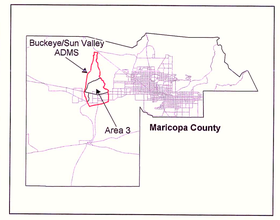
INDEX MAP



N

0 2,000 4,000 6,000 Feet

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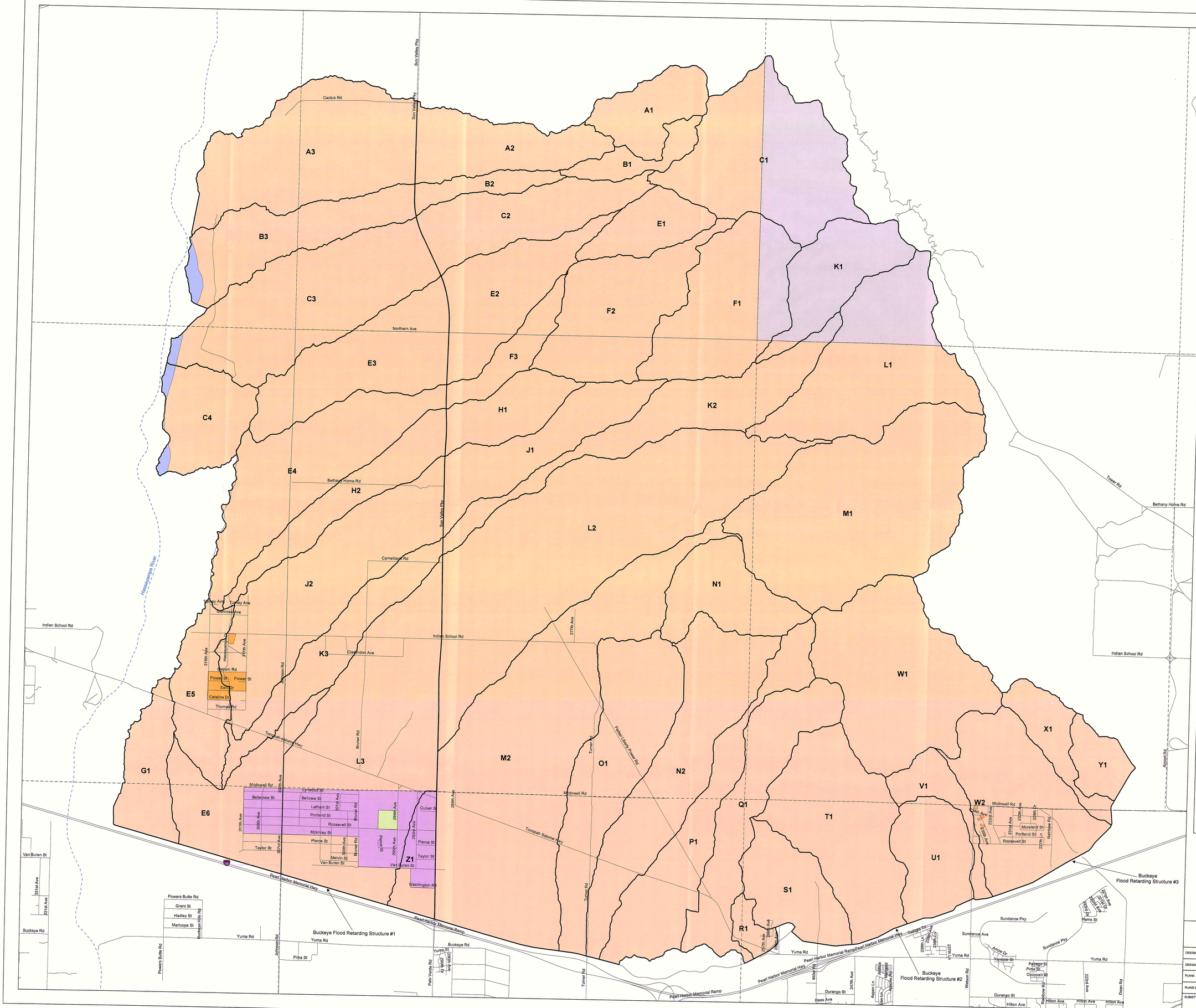

Flood Control District of Maricopa County

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Figure 4-4A
Area 3 Existing Conditions Land Use Map

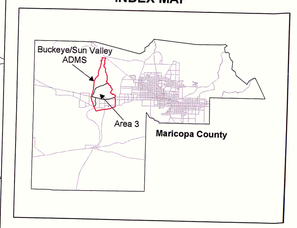
LEGEND

- E1 Sub-Basin Boundary
- Road
- *Estate Residential*
- *Large Lot Residential - Single Family*
- *Transportation*
- *Active Open Space (Includes parks)*
- *Water*
- *Agriculture*
- *Vacant*
- Township-Range

NOTES

Registered Professional Engineer (Civil) CERTIFICATE NO. 42881 STEPHEN W. ROGERS Arizona U.S.A.

INDEX MAP



0 2,000 4,000 6,000 Feet
1 inch equals 2,000 feet

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Horizontal Coordinate System:
NAD 1983 State Plane Arizona Central
Vertical Coordinate System:
NAVD 1988

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PLANS CHK.		
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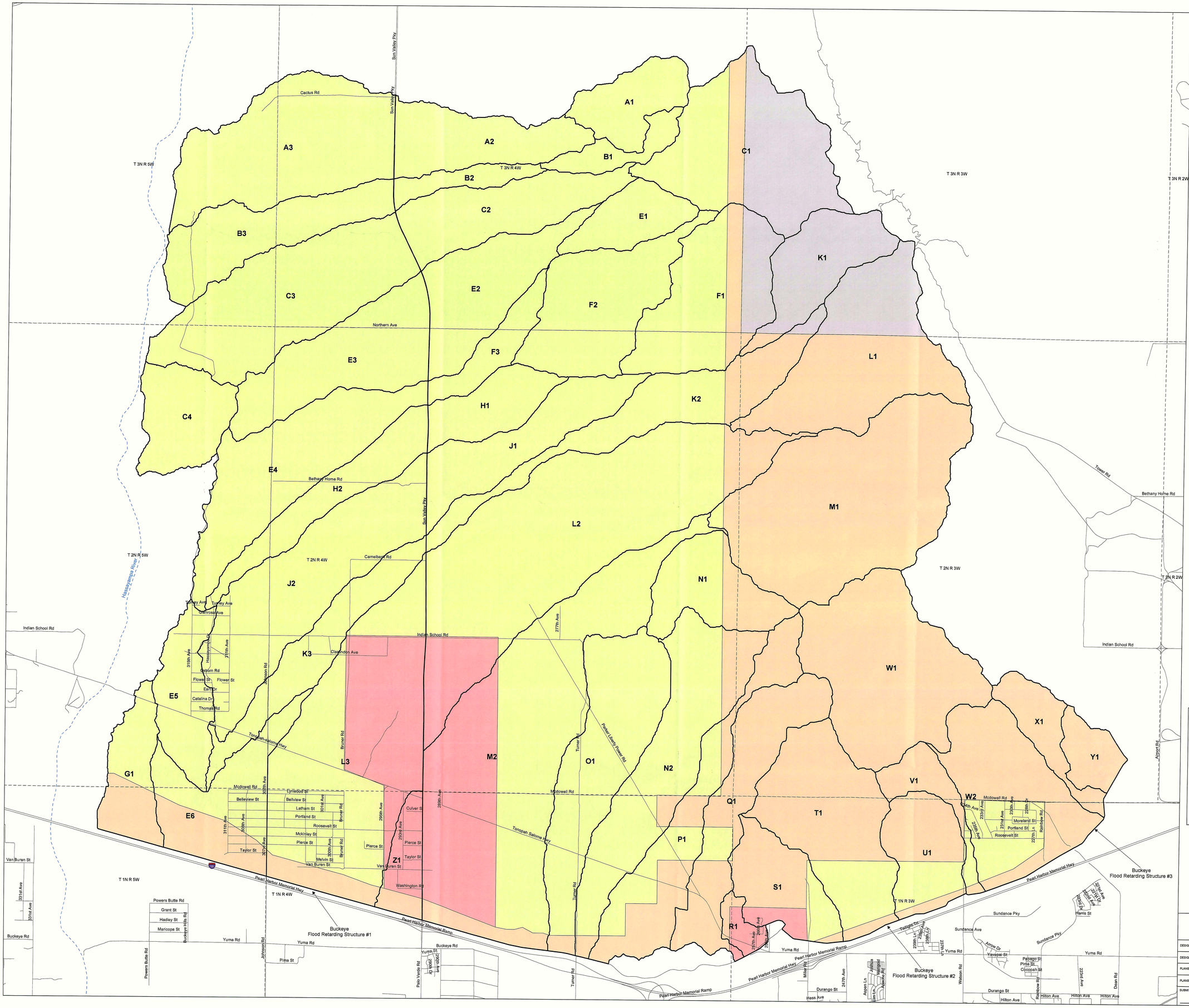
CHIEF ENGINEER AND GENERAL MANAGER

SHEET 1 OF 1



BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY
FCD CONTRACT NO. 2002C027

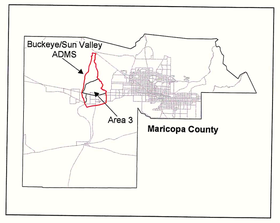
Figure 4-4B
Area 3 Future Conditions Land Use Map

LEGEND

- E1 Sub-Basin Boundary
- Road
- *Central, general, office, intermediate commerce*
- *Medium Density Residential (6K-12K sq ft lot size)*
- *Active Open Space (Includes parks)*
- *Vacant*
- Township - Range

NOTES

INDEX MAP



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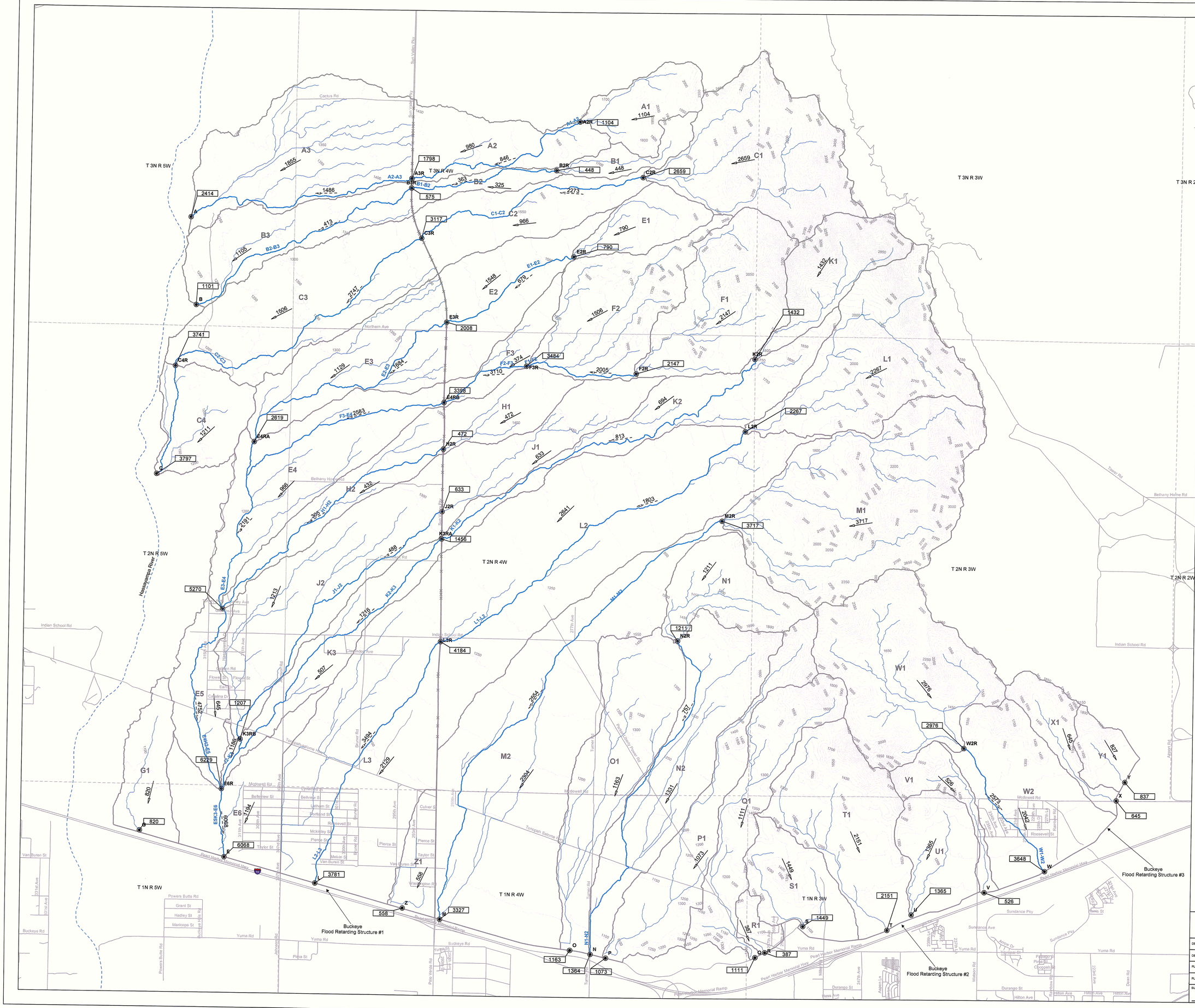


Flood Control District of Maricopa County

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY
FCD CONTRACT NO. 2002C027

Figure 4-5A
Area 3 Existing Conditions Flow Map

LEGEND

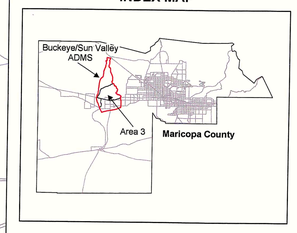
- E1 — Sub-Basin Boundary
- E1 — Concentration Point
- E1-E2 — Routing Channel
- Lag Time Path
- Culvert
- Contour (50ft)
- Township - Range
- Road
- 1254 — Existing 100-year 24-hour Combined Peak Flow (CFS)
- 548 — Existing 100-year 24-hour Sub-Basin Peak Flow (CFS)
- 674 — Existing 100-year 24-hour Routed Peak Flow (CFS)

NOTES

Please refer to Volume V-A Area 3 Hydrology Report for all other peak flows.

Registered Professional Engineer (Civil) CERTIFICATE NO. 42981 STEPHEN W. ROGERS Date Signed 7/15/05

INDEX MAP



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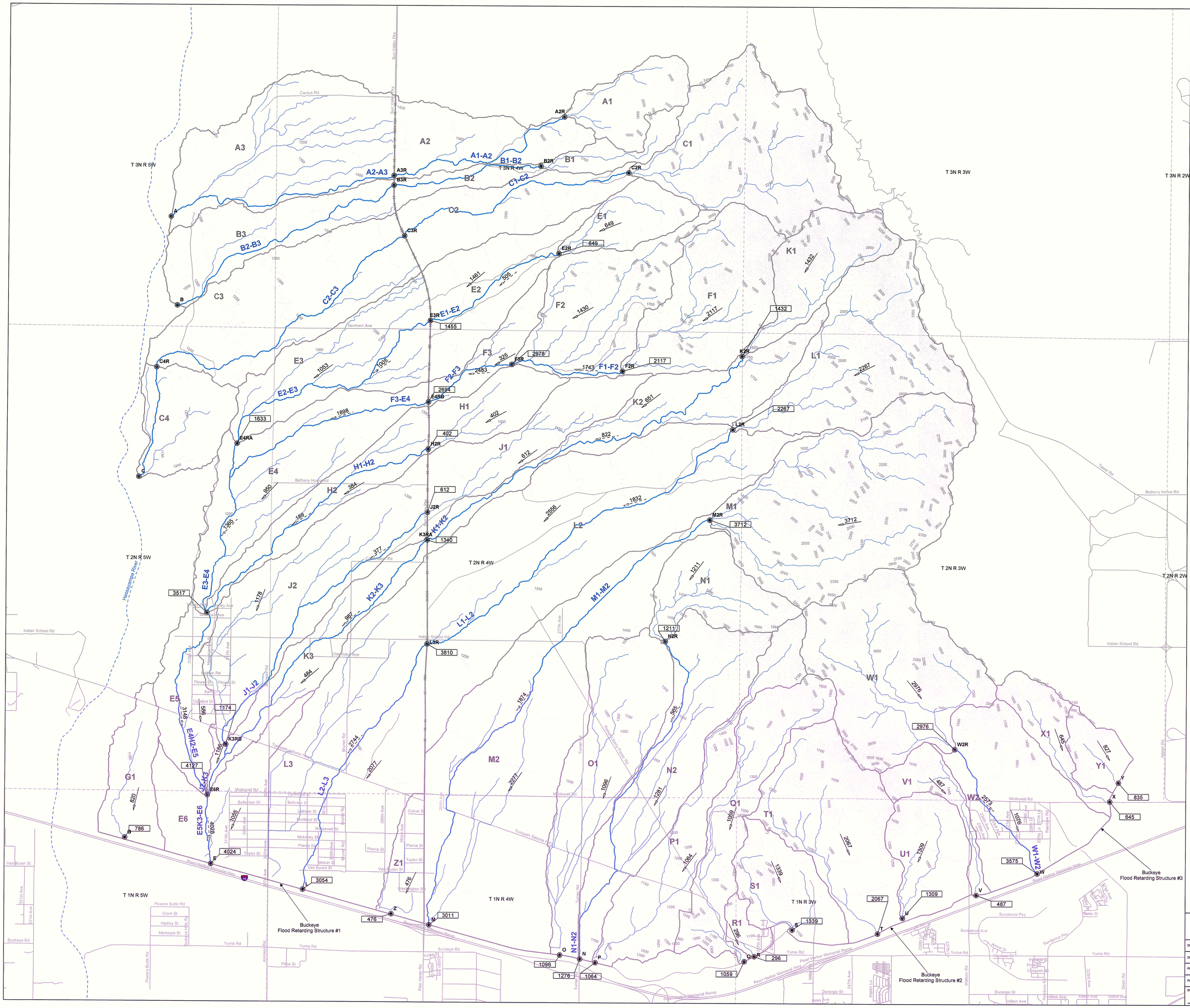


BUCKEYE/SUN VALLEY AREA DRAINAGE MASTER STUDY
FCD CONTRACT NO. 2002C027
Figure 4-5B
Area 3 Future Conditions Flow Map

LEGEND

- E1 — Sub-Basin Boundary
- E — Concentration Point
- E1-E2 — Routing Channel
- Lag Time Path
- Culvert
- Contour (50ft)
- Township - Range
- Road
- 1254 — Future 100-year 24-hour Combined Peak Flow (CFS)
- 548 — Future 100-year 24-hour Sub-Basin Peak Flow (CFS)
- 674 — Future 100-year 24-hour Routed Peak Flow (CFS)

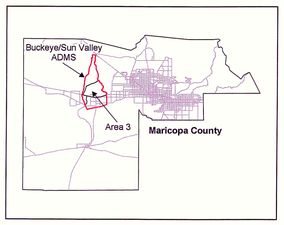
NOTES

Sub-basins A-C are not included in Area 3. However, the existing hydrologic conditions were simulated for sub-basins A-C to update Alpha Engineering's previous study (1996).

Please to Volume V-A Area 3 Hydrology Report for all other peak flows.

Registered Professional Engineer (Civil) CERTIFICATE NO. 42881 STEPHEN W. ROGERS Date Signed 7/1/05

INDEX MAP



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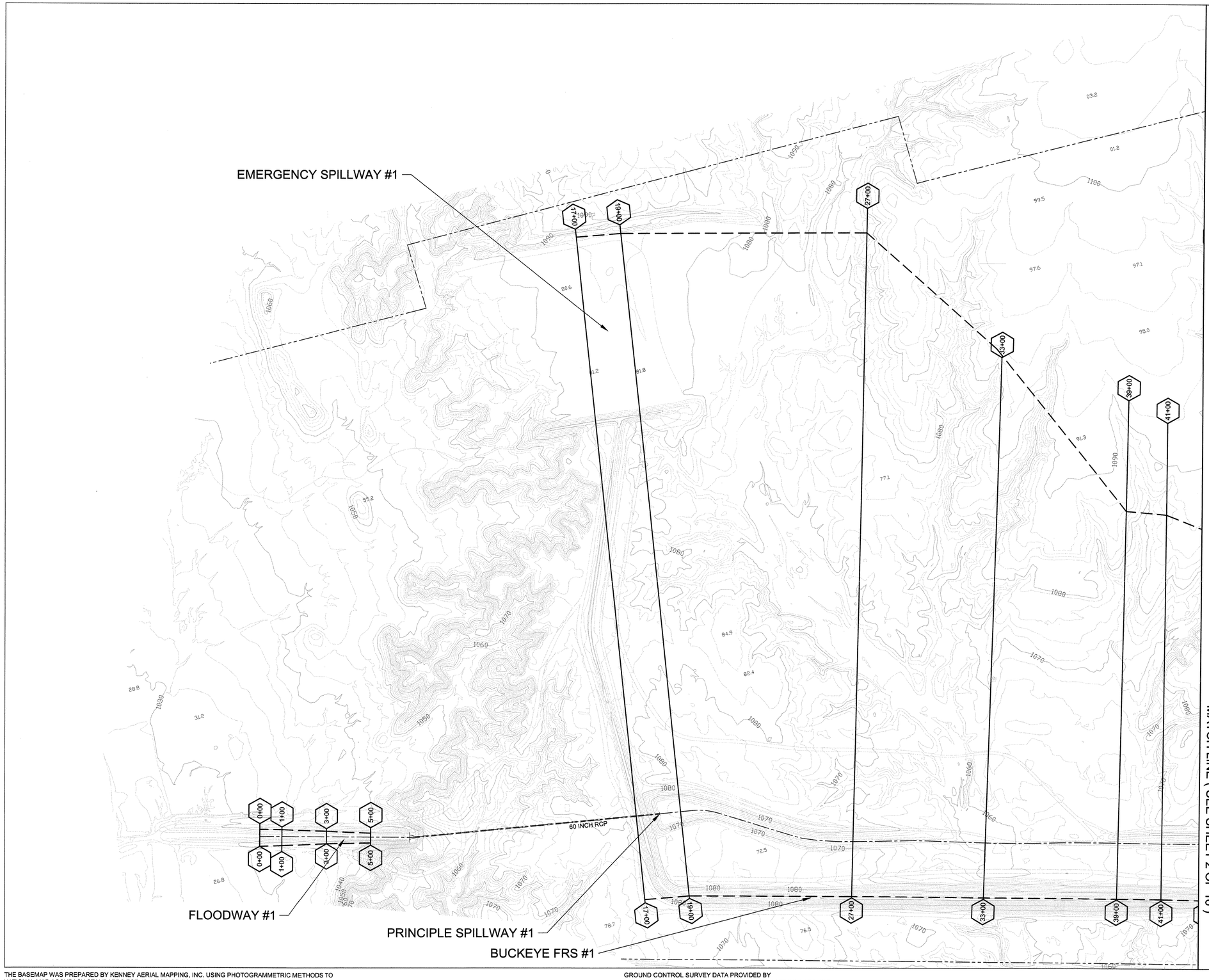

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION — 2+00 — 2+00

PMF BOUNDARY

HYDRAULIC CENTERLINE

FCDMC PROPERTY LINE

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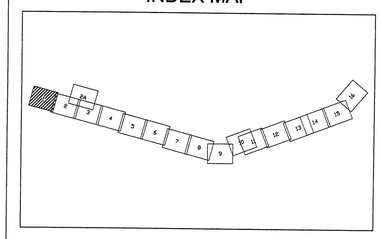

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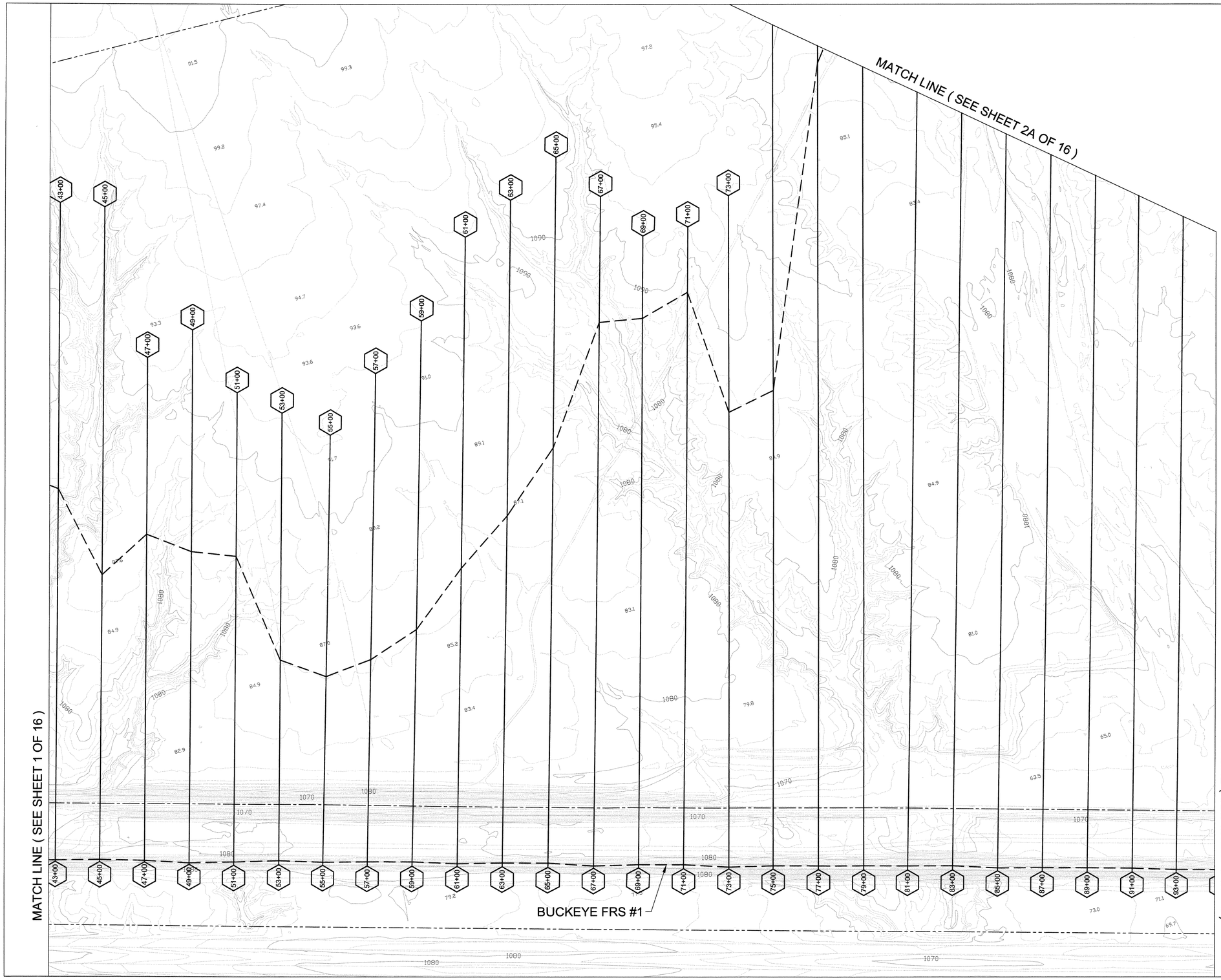

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION	2+00 — 2+00
PMF BOUNDARY	
HYDRAULIC CENTERLINE	
FCDMC PROPERTY LINE	

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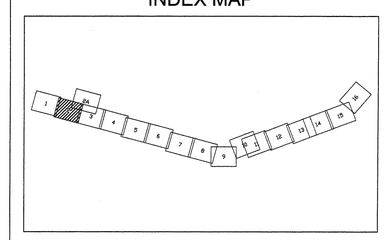

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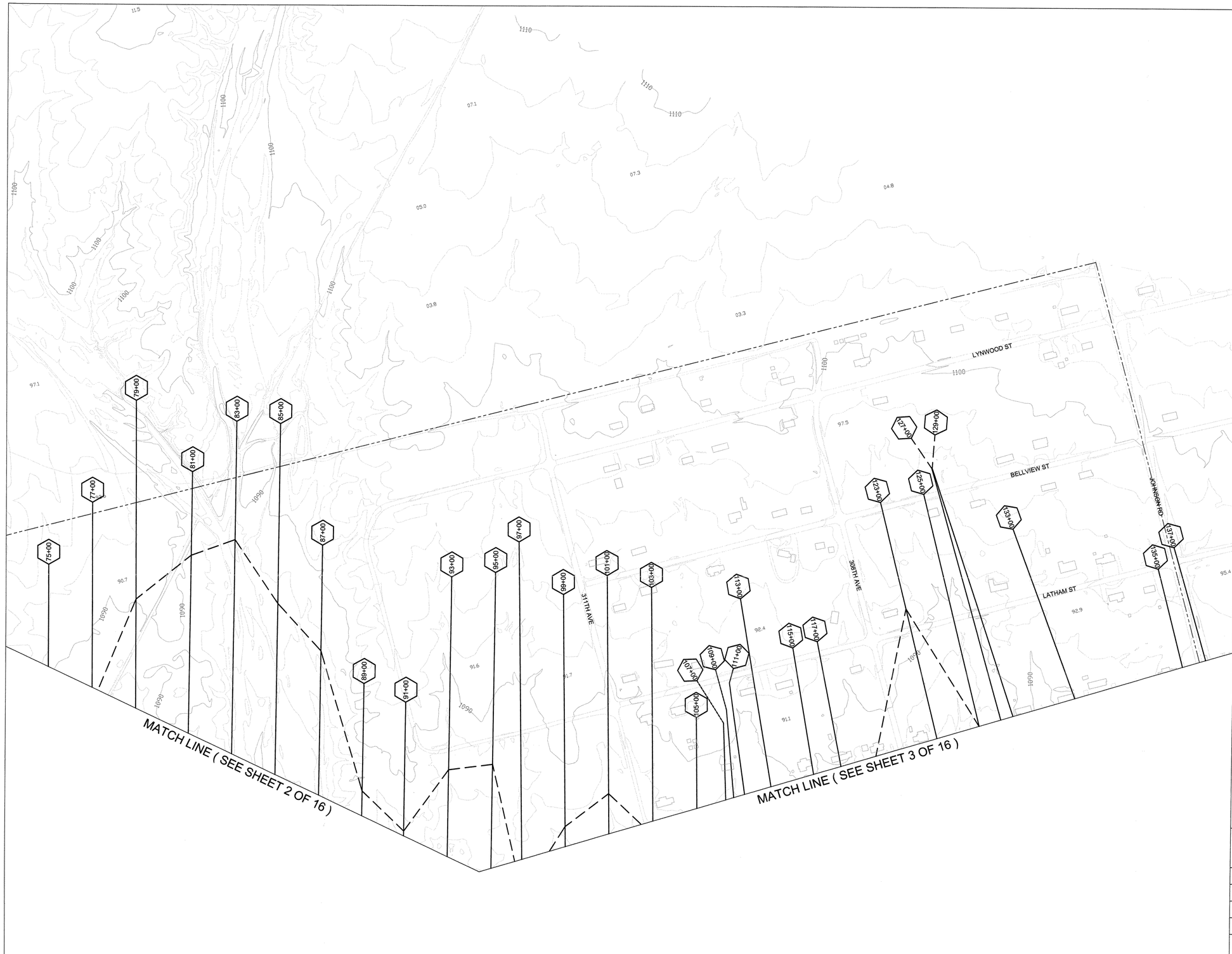

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

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LEGEND

CROSS SECTION LOCATION	2+00 — 2+00
PMF BOUNDARY	
HYDRAULIC CENTERLINE	
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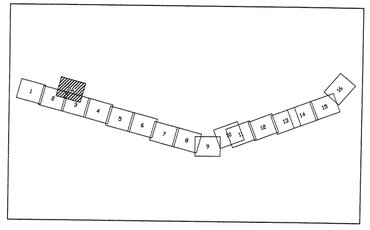

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SHEET 2A OF 16

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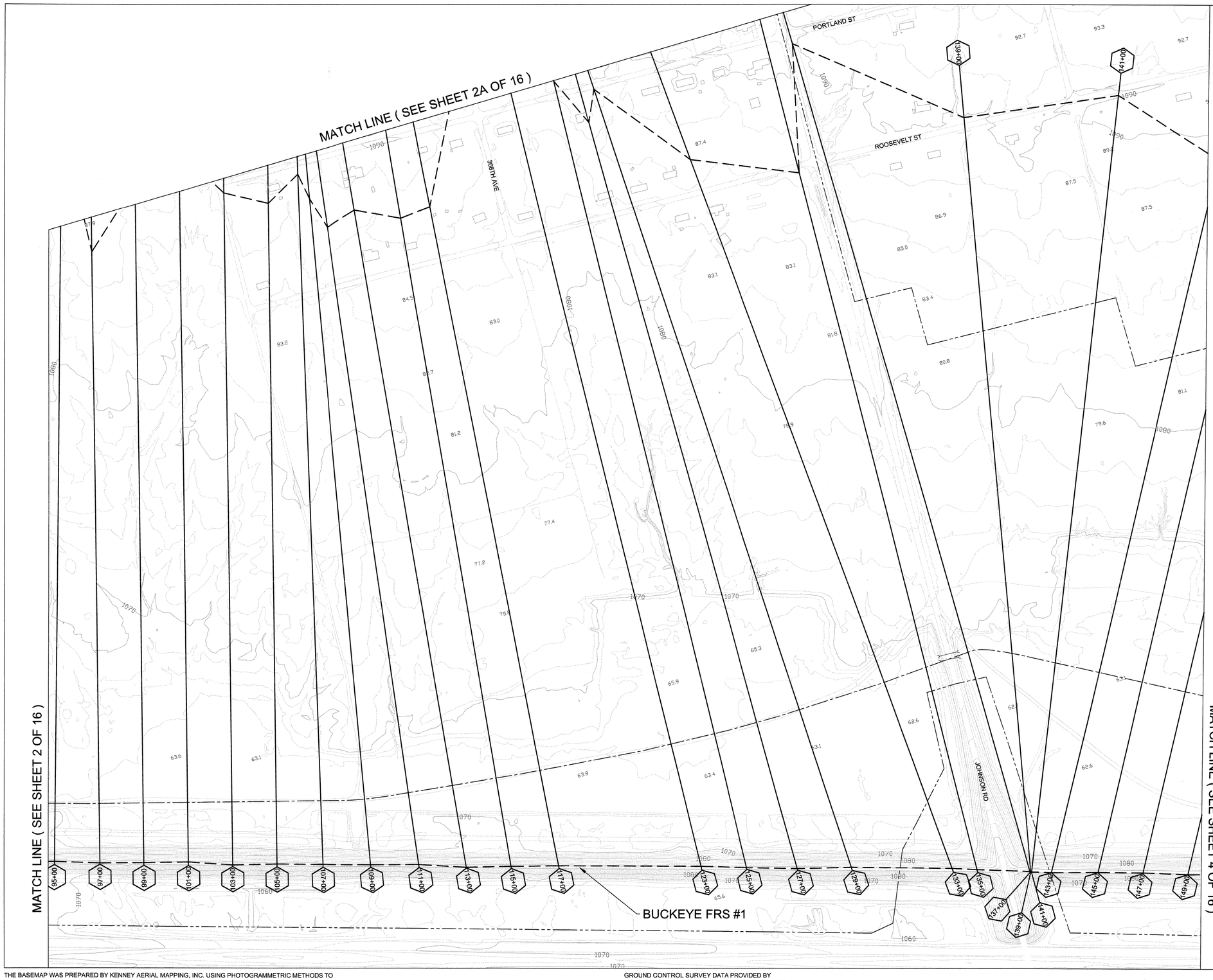

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

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CROSS SECTION LOCATION	2+00 — 2+00
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FCDMC PROPERTY LINE	

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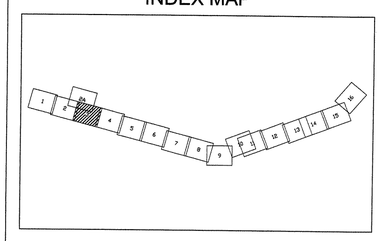

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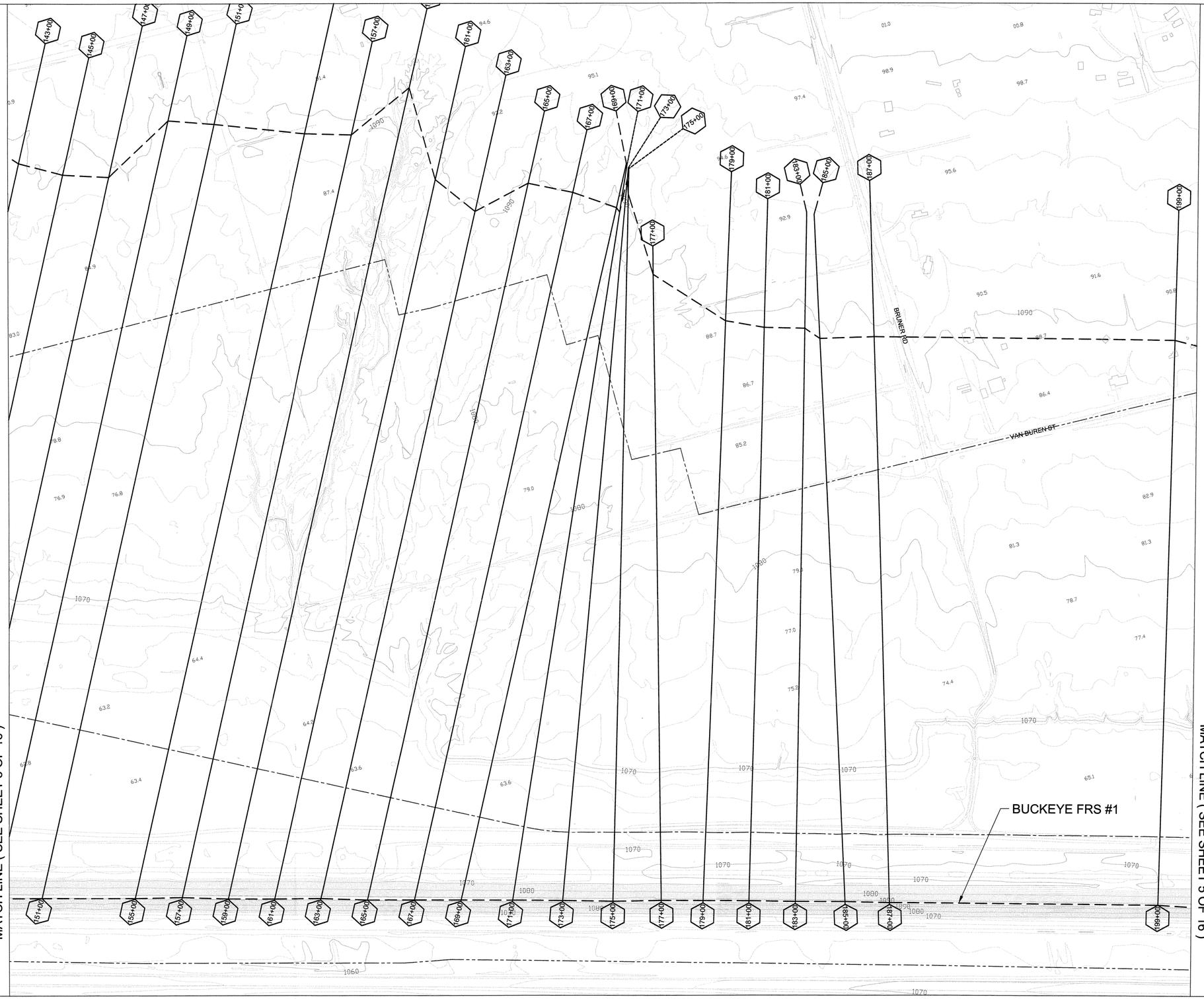

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

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BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

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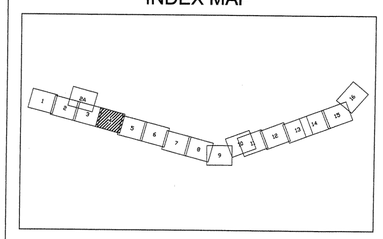

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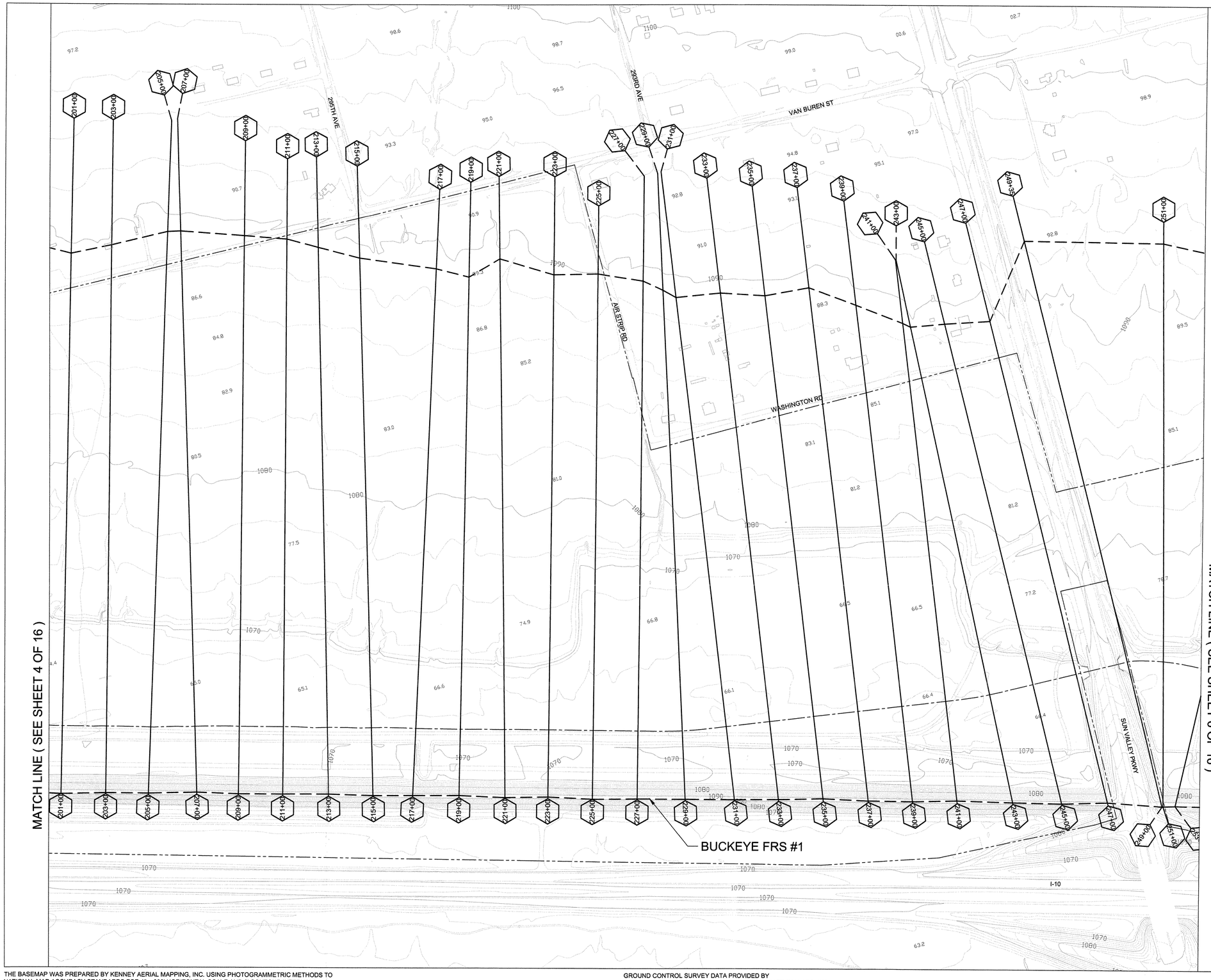

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BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

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HEC-RAS CROSS-SECTION LOCATION MAP

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CROSS SECTION LOCATION — 2+00 — 2+00

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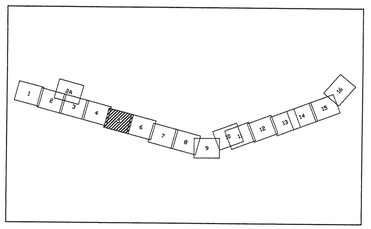

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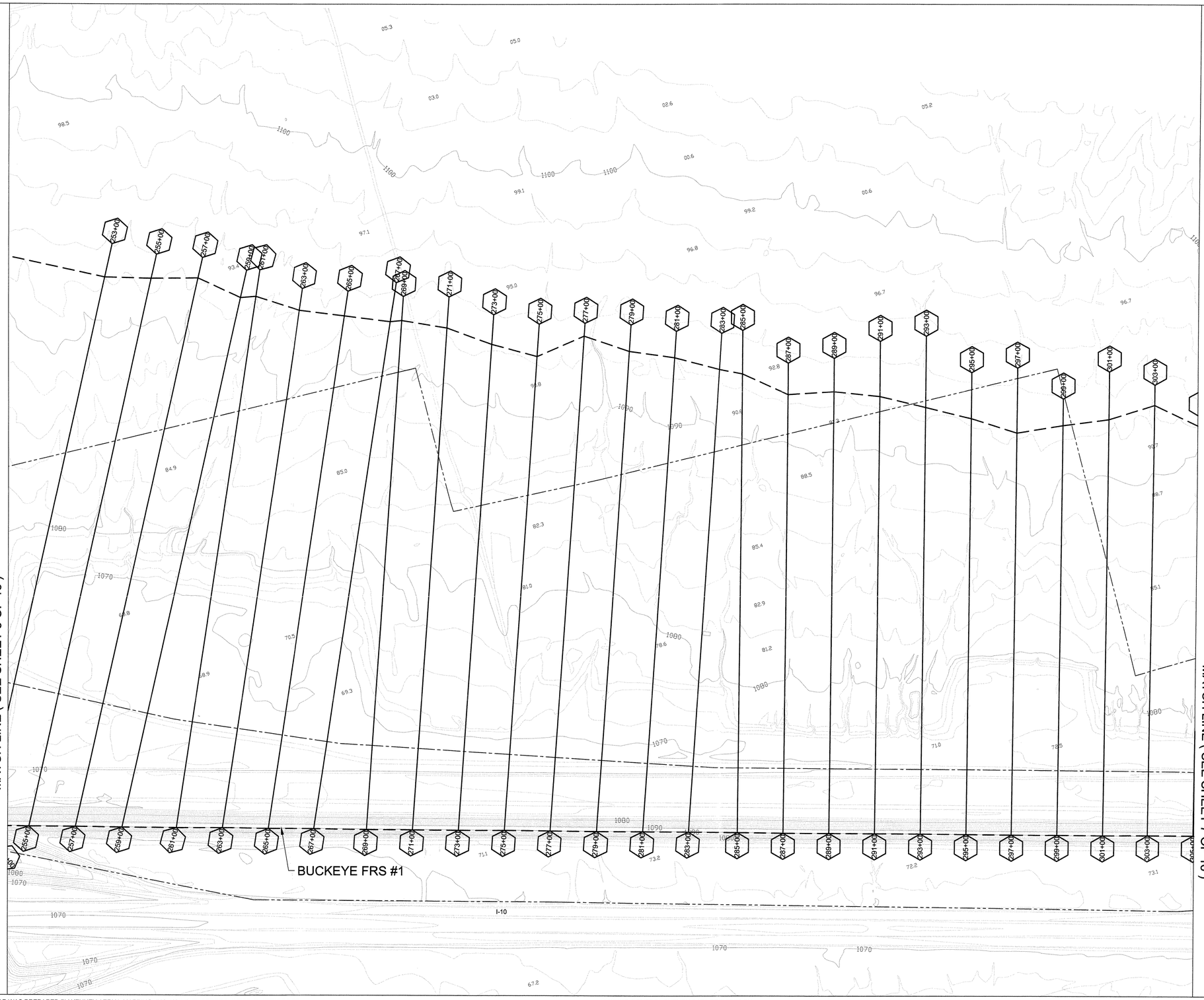

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

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LEGEND

CROSS SECTION LOCATION 2+00 — 2+00

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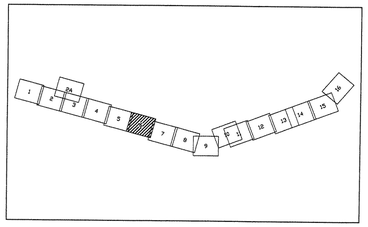

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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION 2+00 — 2+00

PMF BOUNDARY

HYDRAULIC CENTERLINE

FCDMC PROPERTY LINE

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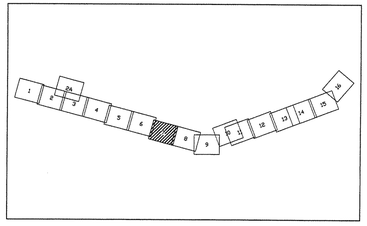

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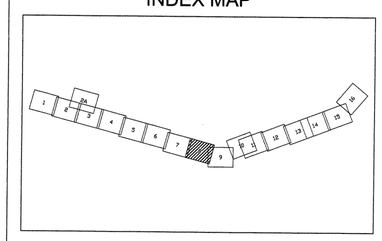


Registered Professional Engineer (Civil)
CERTIFICATE NO. 42881
STEPHEN W. ROGERS
Date Signed 7-13-04
ARIZONA U.S.A.

N

200 0 200 400

SCALE: 1" = 200'
CONTOUR INTERVAL = 2 FEET

PBSJ
7310 N. 16th St., Suite 310
Phoenix, Arizona 85020
Phone: (602) 943-1003
Fax: (602) 943-1303

	BY	DATE
DESIGN	JSL	4/04
DESIGN CHK.	WC	4/04
PLANS		
PLANS CHK.		
SUBMITTED BY:		DATE

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

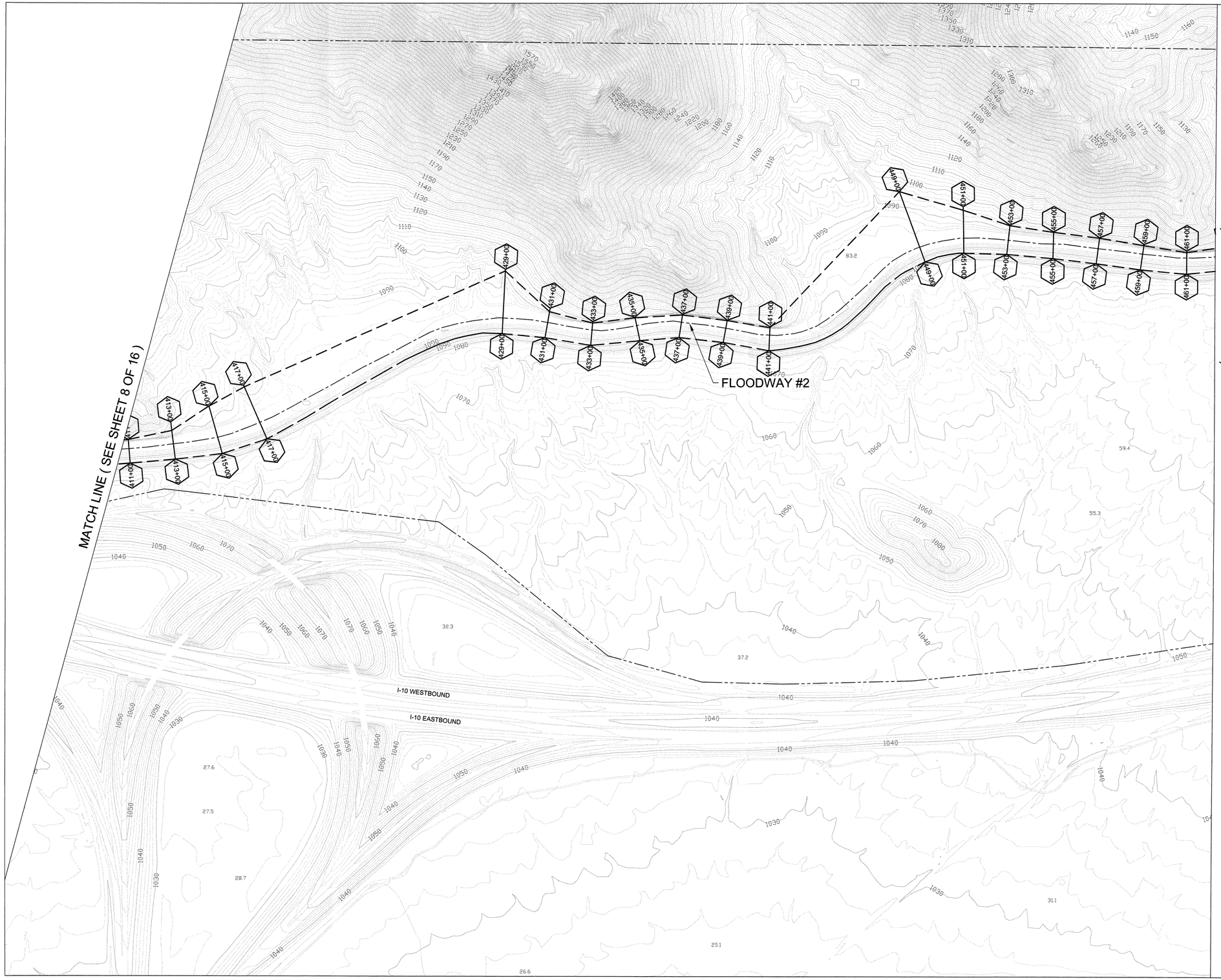
APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 8 OF 16

THE BASEMAP WAS PREPARED BY KENNEY AERIAL MAPPING, INC. USING PHOTOGRAMMETRIC METHODS TO NATIONAL MAP ACCURACY STANDARDS FOR 1" = 200' HORIZONTAL SCALE AND 2' CONTOUR INTERVALS.

GROUND CONTROL SURVEY DATA PROVIDED BY
PROJECT ENGINEERING CONSULTANTS, LTD. & HUITT-ZOLLARS, INC.



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GROUND CONTROL SURVEY DATA PROVIDED BY PROJECT ENGINEERING CONSULTANTS, LTD. & HUITT-ZOLLARS, INC.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION — 2+00 — 2+00

PMF BOUNDARY

HYDRAULIC CENTERLINE

FCDMC PROPERTY LINE

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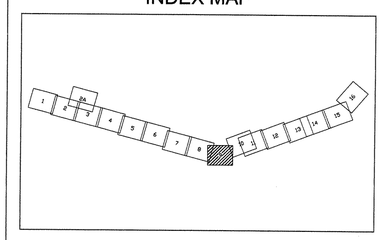
INDEX MAP



N

200 0 200 400

SCALE: 1" = 200'
CONTOUR INTERVAL = 2 FEET

Registered Professional Surveyor (Civil)
CERTIFICATE NO. 42881
STEPHEN W. ROGERS
Date Signed 4/26/04
ARIZONA U.S.A.

PBSJ
7310 N. 16th St., Suite 310
Phoenix, Arizona 85020
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Fax: (602) 943-1303

	BY	DATE
DESIGN	JSL	4/04
DESIGN CHK.	WC	4/04
PLANS		
PLANS CHK.		

SUBMITTED BY: DATE

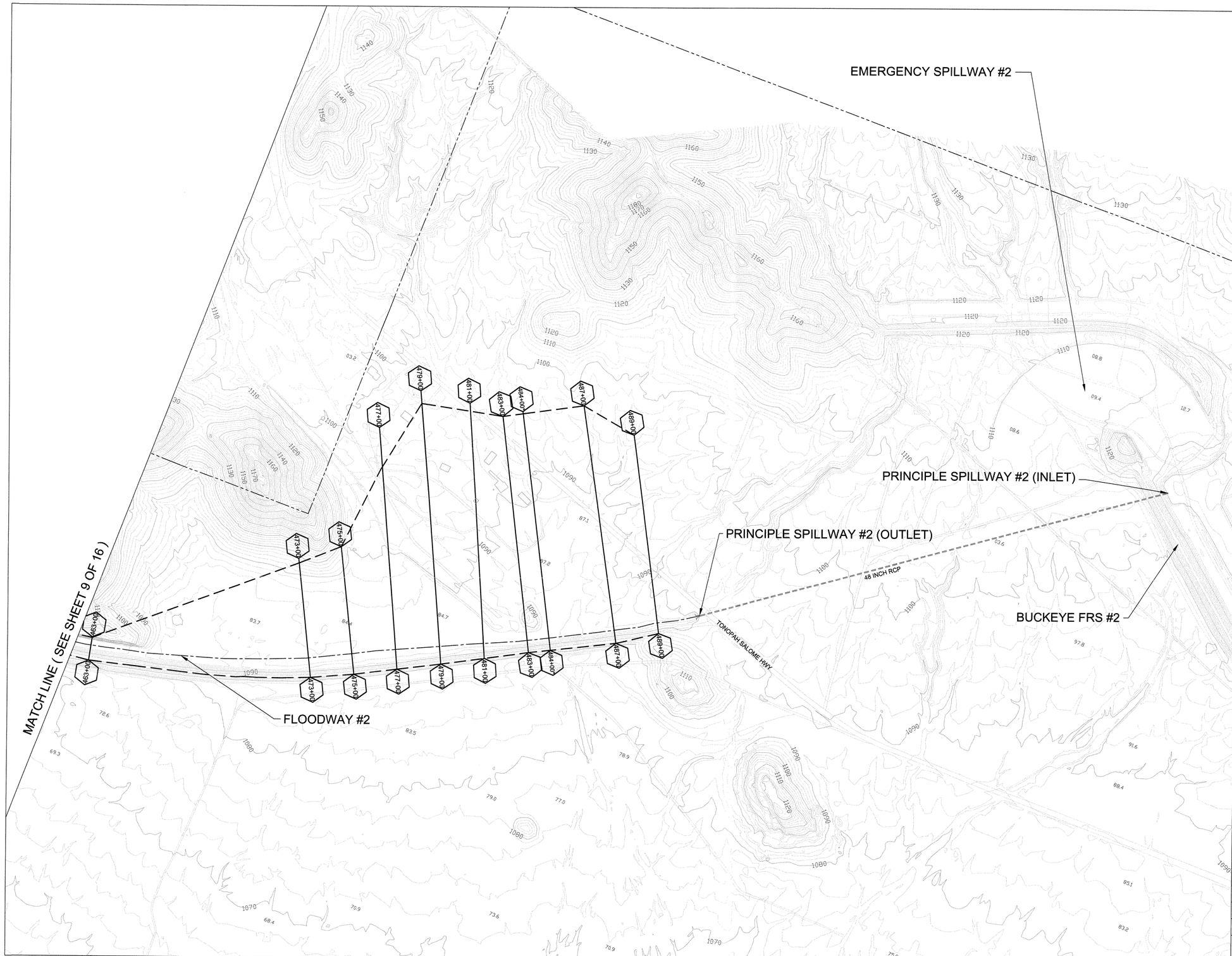
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 9 OF 16



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION	2+00 — 2+00
PMF BOUNDARY	
HYDRAULIC CENTERLINE	
FCDMC PROPERTY LINE	

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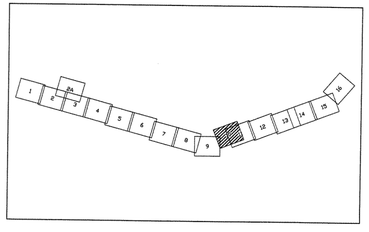

Registered Professional Engineer (Civil) CERTIFICATE NO. 42881 STEPHEN W. ROGERS Date Signed 7/1/04

N

200 0 200 400

SCALE: 1" = 200'
CONTOUR INTERVAL = 2 FEET

PBSJ
7310 N. 16th St., Suite 310
Phoenix, Arizona 85020
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Fax: (602) 943-1303

	BY	DATE
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DESIGN CHK.	WC	4/04
PLANS		
PLANS CHK.		
SUBMITTED BY: DATE		

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 10 OF 16

THE BASEMAP WAS PREPARED BY KENNEY AERIAL MAPPING, INC. USING PHOTOGRAMMETRIC METHODS TO NATIONAL MAP ACCURACY STANDARDS FOR 1" = 200' HORIZONTAL SCALE AND 2' CONTOUR INTERVALS.

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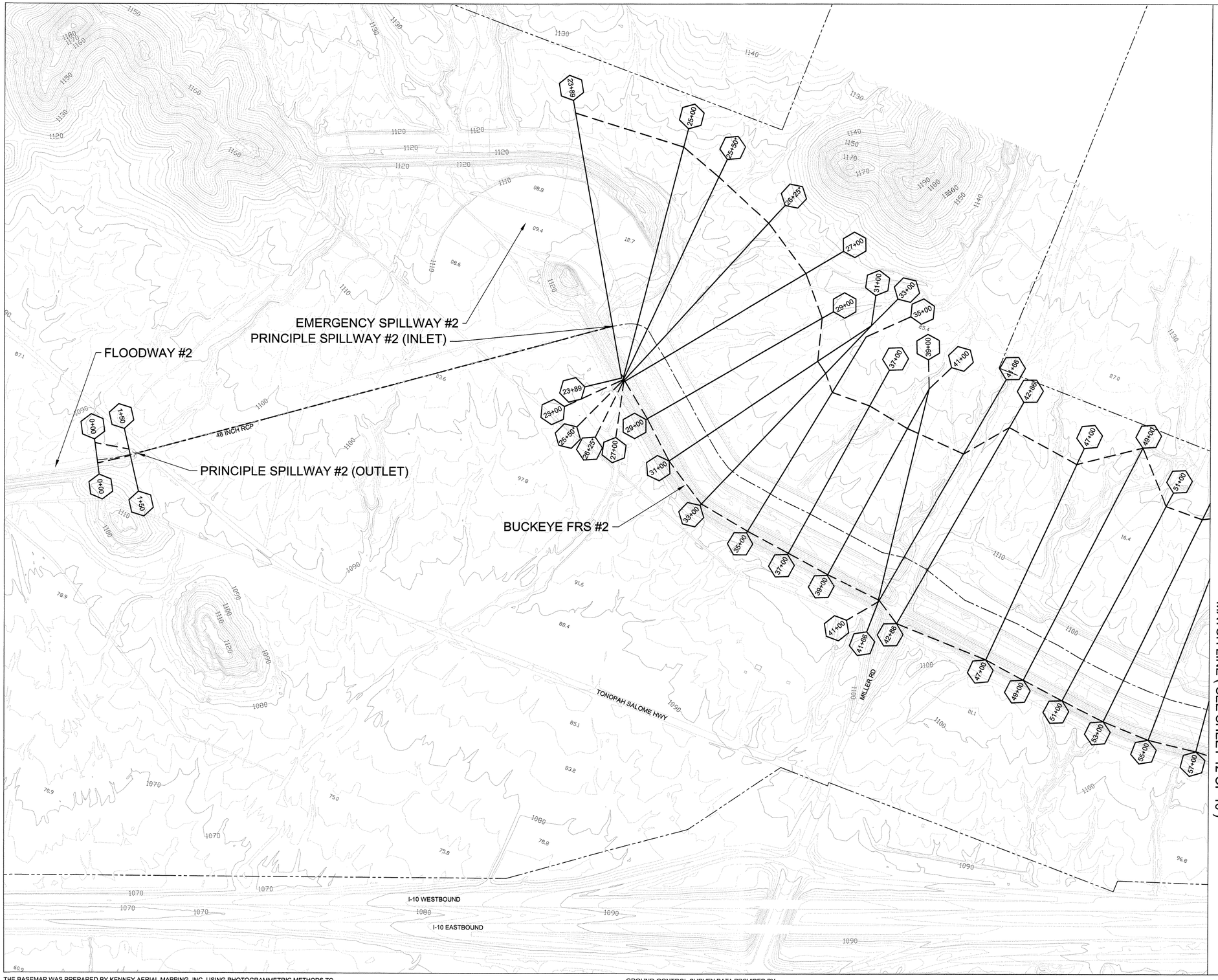

MATCH LINE (SEE SHEET 12 OF 16)

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION	2+00 — 2+00
PMF BOUNDARY	
HYDRAULIC CENTERLINE	
FCDMC PROPERTY LINE	

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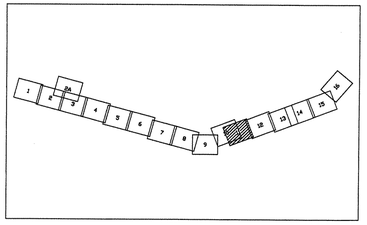

Registered Professional Engineer (Civil)
CERTIFICATE NO. 42881
STEPHEN W. ROGERS
Date Signed 7-1-04
ARIZONA U.S.A.

N

200 0 200 400

SCALE: 1" = 200'
CONTOUR INTERVAL = 2 FEET

PBSJ
7310 N. 16th St., Suite 310
Phoenix, Arizona 85020
Phone: (602) 943-1003
Fax: (602) 943-1303

	BY	DATE
DESIGN	JSL	4/04
DESIGN CHK.	WC	4/04
PLANS		
PLANS CHK.		
SUBMITTED BY: DATE		

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 11 OF 16

THE BASEMAP WAS PREPARED BY KENNEY AERIAL MAPPING, INC. USING PHOTOGRAMMETRIC METHODS TO NATIONAL MAP ACCURACY STANDARDS FOR 1" = 200' HORIZONTAL SCALE AND 2' CONTOUR INTERVALS.

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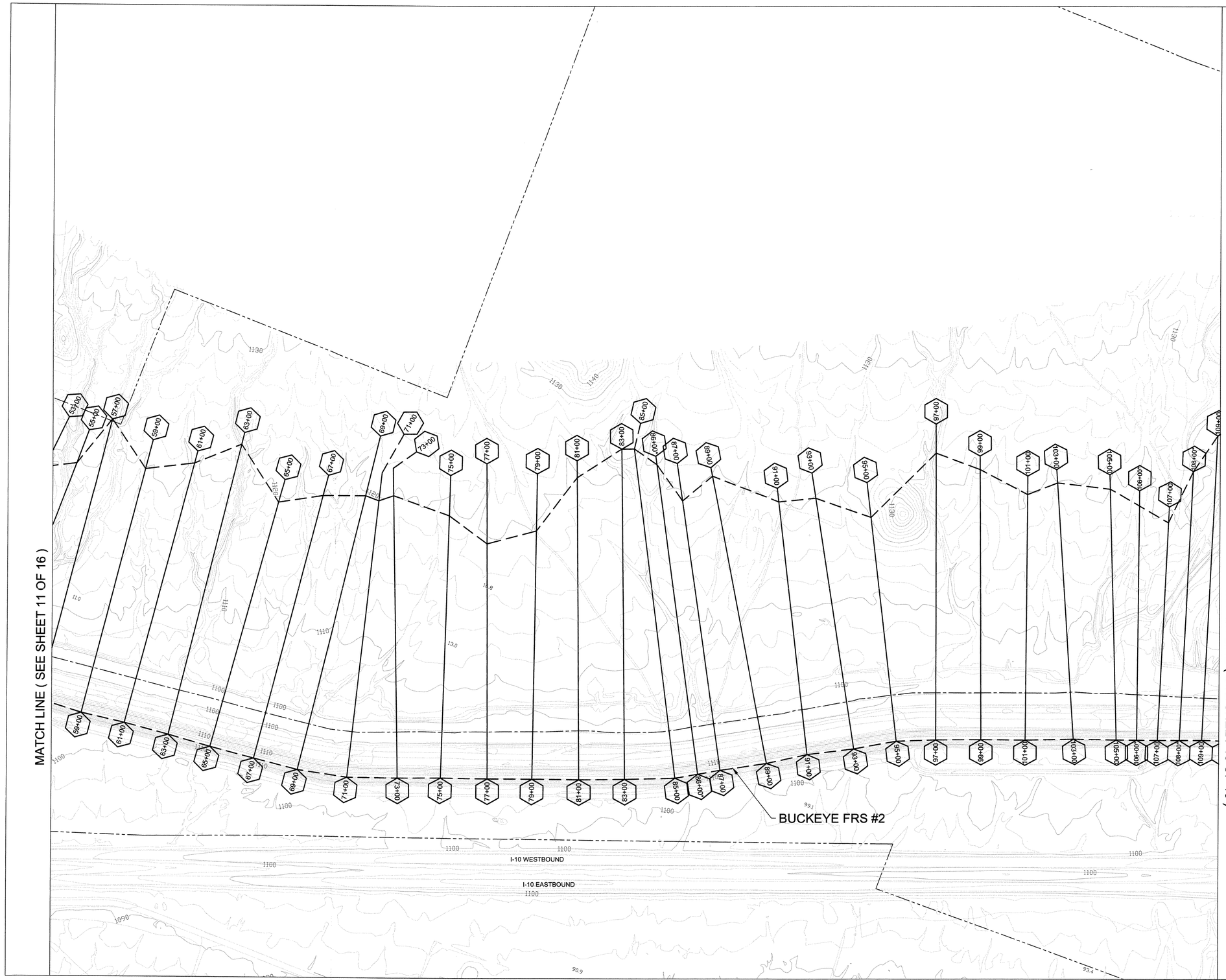

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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION — 2+00 — 2+00

PMF BOUNDARY

HYDRAULIC CENTERLINE

FCDMC PROPERTY LINE

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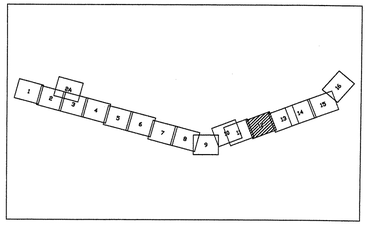

Registered Professional Engineer (Civil) CERTIFICATE NO. 42881 STEPHEN W. ROGERS Date Signed 7/1/04 ARIZONA U.S.A.

N

200 0 200 400

SCALE: 1" = 200'
CONTOUR INTERVAL = 2 FEET

PBSJ
7310 N. 16th St., Suite 310
Phoenix, Arizona 85020
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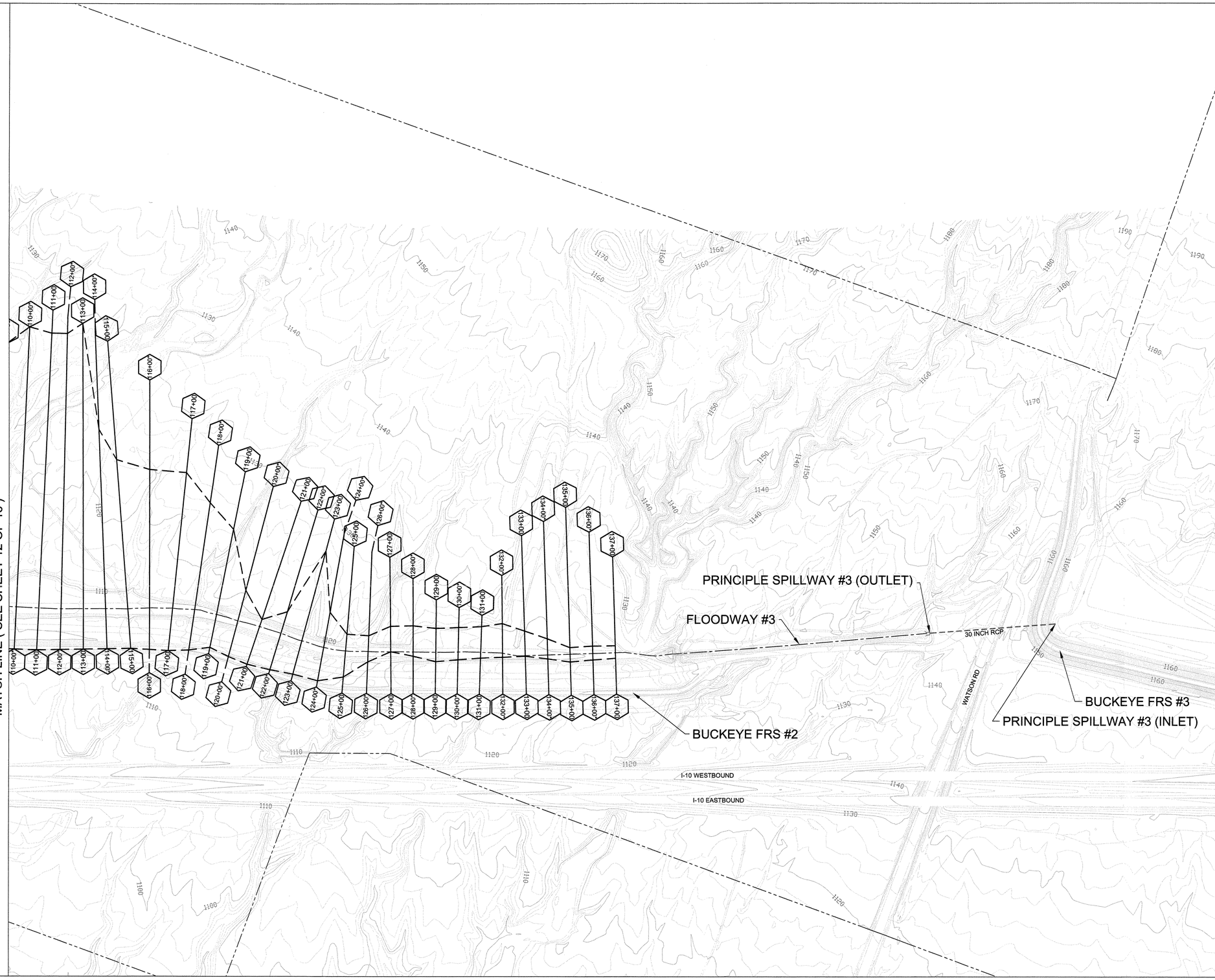
	BY	DATE
DESIGN	JSL	4/04
DESIGN CHK.	WC	4/04
PLANS		
PLANS CHK.		
SUBMITTED BY:		DATE

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER



FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION	2+00 — 2+00
PMF BOUNDARY	
HYDRAULIC CENTERLINE	
FCDMC PROPERTY LINE	

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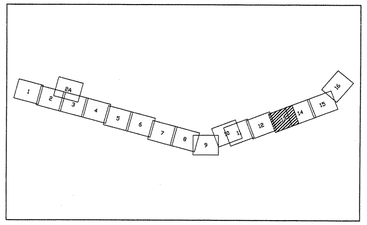

Registered Professional Engineer (Civil) CERTIFICATE NO. 42881 STEPHEN W. ROGERS Date Signed 7/16/04 ARIZONA U.S.A.

N

200 0 200 400

SCALE: 1" = 200'
CONTOUR INTERVAL = 2 FEET

PBSJ
7310 N. 16th St., Suite 310
Phoenix, Arizona 85020
Phone: (602) 943-1003
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	BY	DATE
DESIGN	JSL	4/04
DESIGN CHK.	WC	4/04
PLANS		
PLANS CHK.		
SUBMITTED BY: DATE		

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 13 OF 16

THE BASEMAP WAS PREPARED BY KENNEY AERIAL MAPPING, INC. USING PHOTOGRAMMETRIC METHODS TO NATIONAL MAP ACCURACY STANDARDS FOR 1" = 200' HORIZONTAL SCALE AND 2' CONTOUR INTERVALS.

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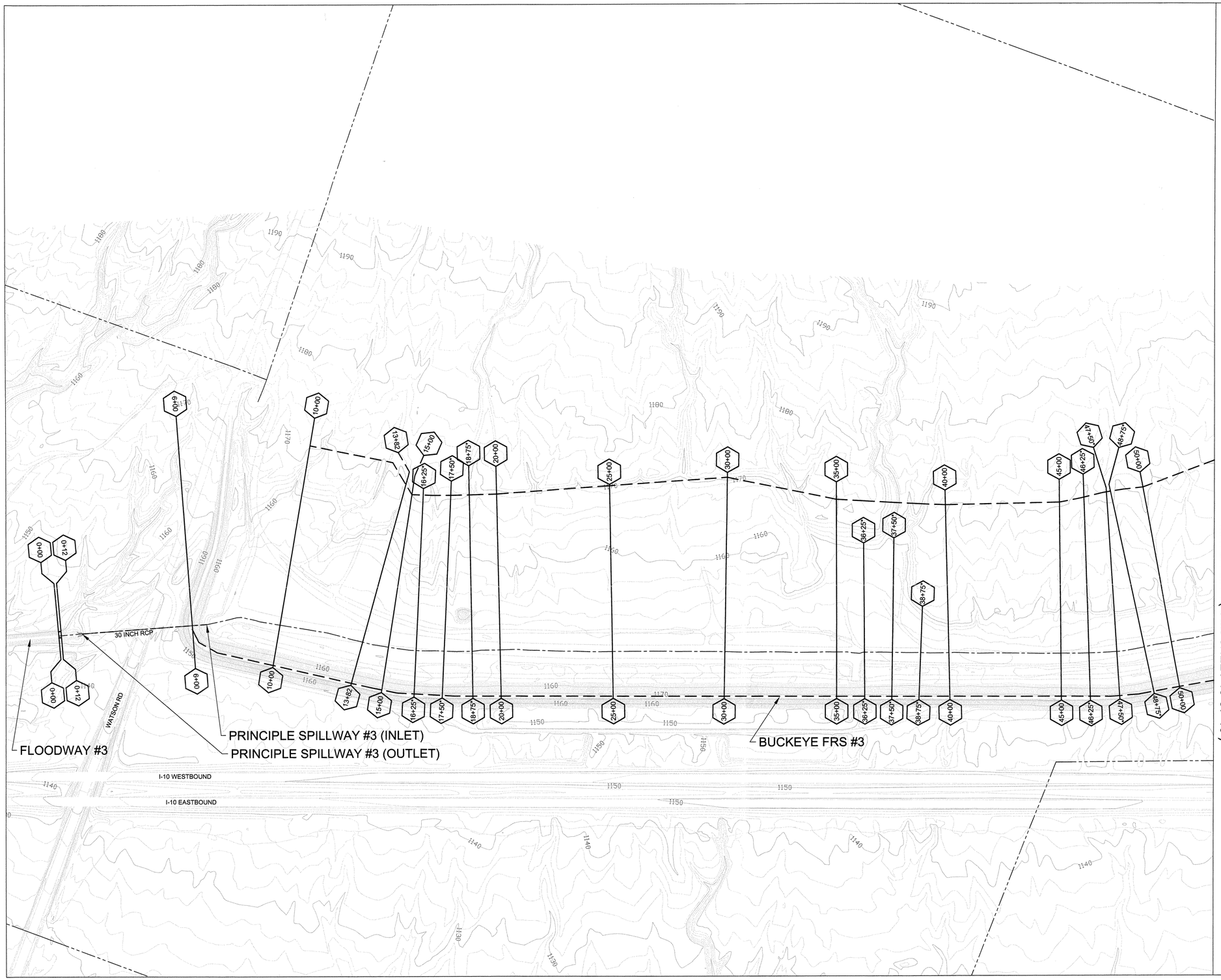

MATCH LINE (SEE SHEET 15 OF 16)

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION 2+00 — 2+00

PMF BOUNDARY

HYDRAULIC CENTERLINE

FCDMC PROPERTY LINE

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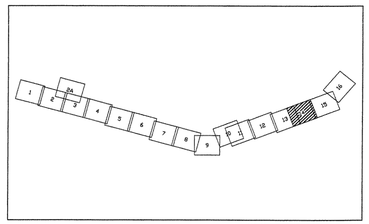

Registered Professional Engineer (Civil) CERTIFICATE NO. 42881 STEPHEN W. ROGERS Date Signed 7/1/05

N

200 0 200 400

SCALE: 1" = 200'
CONTOUR INTERVAL = 2 FEET

PBSJ
7310 N. 16th St., Suite 310
Phoenix, Arizona 85020
Phone: (602) 943-1003
Fax: (602) 943-1303

	BY	DATE
DESIGN	JSL	4/04
DESIGN CHK.	WC	4/04
PLANS		
PLANS CHK.		
SUBMITTED BY:		DATE

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 14 OF 16

THE BASEMAP WAS PREPARED BY KENNEY AERIAL MAPPING, INC. USING PHOTOGRAMMETRIC METHODS TO NATIONAL MAP ACCURACY STANDARDS FOR 1" = 200' HORIZONTAL SCALE AND 2' CONTOUR INTERVALS.

GROUND CONTROL SURVEY DATA PROVIDED BY
PROJECT ENGINEERING CONSULTANTS, LTD. & HUITT-ZOLLARS, INC.

MATCH LINE (SEE SHEET 14 OF 16)

MATCH LINE (SEE SHEET 16 OF 16)

BUCKEYE FRS #3

I-10 WESTBOUND

I-10 EASTBOUND

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION	2+00 — 2+00
PMF BOUNDARY	
HYDRAULIC CENTERLINE	
FCDMC PROPERTY LINE	

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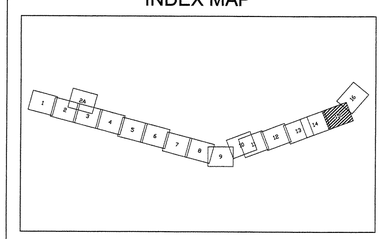

Registered Professional Engineer (Civil)
CERTIFICATE NO. 42881
STEPHEN W. ROGERS
Date Signed 7-02-04
ARIZONA U.S.A.

N

200 0 200 400

SCALE: 1" = 200'
CONTOUR INTERVAL = 2 FEET

PBSJ
7310 N. 16th St., Suite 310
Phoenix, Arizona 85020
Phone: (602) 943-1003
Fax: (602) 943-1303

	BY	DATE
DESIGN	JSL	4/04
DESIGN CHK.	WC	4/04
PLANS		
PLANS CHK.		
SUBMITTED BY: DATE		

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 15 OF 16

THE BASEMAP WAS PREPARED BY KENNEY AERIAL MAPPING, INC. USING PHOTOGRAMMETRIC METHODS TO NATIONAL MAP ACCURACY STANDARDS FOR 1" = 200' HORIZONTAL SCALE AND 2' CONTOUR INTERVALS.

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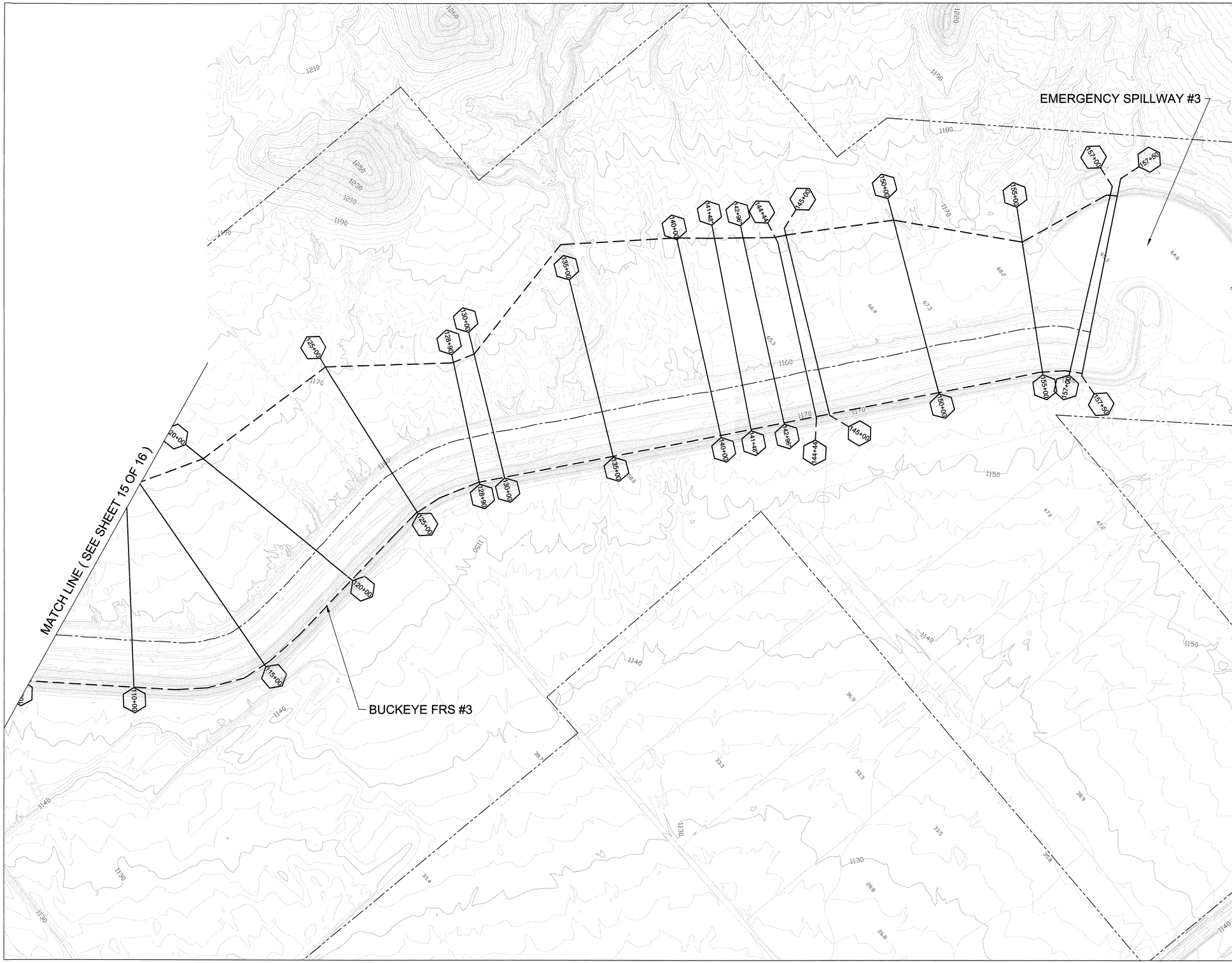

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FCD CONTRACT NO. 2002C027

BUCKEYE/SUN VALLEY
AREA DRAINAGE MASTER STUDY

BUCKEYE FRS
HEC-RAS CROSS-SECTION LOCATION MAP

FINAL

LEGEND

CROSS SECTION LOCATION	2+00 — 2+00
PMF BOUNDARY	
HYDRAULIC CENTERLINE	
FCDMC PROPERTY LINE	

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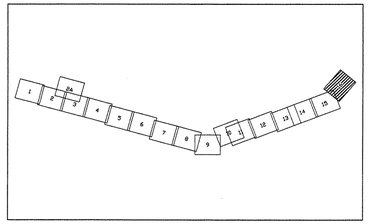
INDEX MAP



Registered Professional Engineer (Civil) CERTIFICATE NO. 42881 STEPHEN W. ROGERS Date Signed 7/08

N

200 0 200 400

SCALE: 1" = 200'
CONTOUR INTERVAL = 2 FEET

PBSJ
7310 N. 16th St., Suite 310
Phoenix, Arizona 85020
Phone: (602) 943-1003
Fax: (602) 943-1303

	BY	DATE
DESIGN	JSL	4/04
DESIGN CHK.	WC	4/04
PLANS		
PLANS CHK.		

SUBMITTED BY: DATE

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

RECOMMENDED BY: DATE

APPROVED BY: DATE

CHIEF ENGINEER AND GENERAL MANAGER

SHEET 16 OF 16

THE BASEMAP WAS PREPARED BY KENNEY AERIAL MAPPING, INC. USING PHOTOGRAMMETRIC METHODS TO NATIONAL MAP ACCURACY STANDARDS FOR 1" = 200' HORIZONTAL SCALE AND 2' CONTOUR INTERVALS.

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