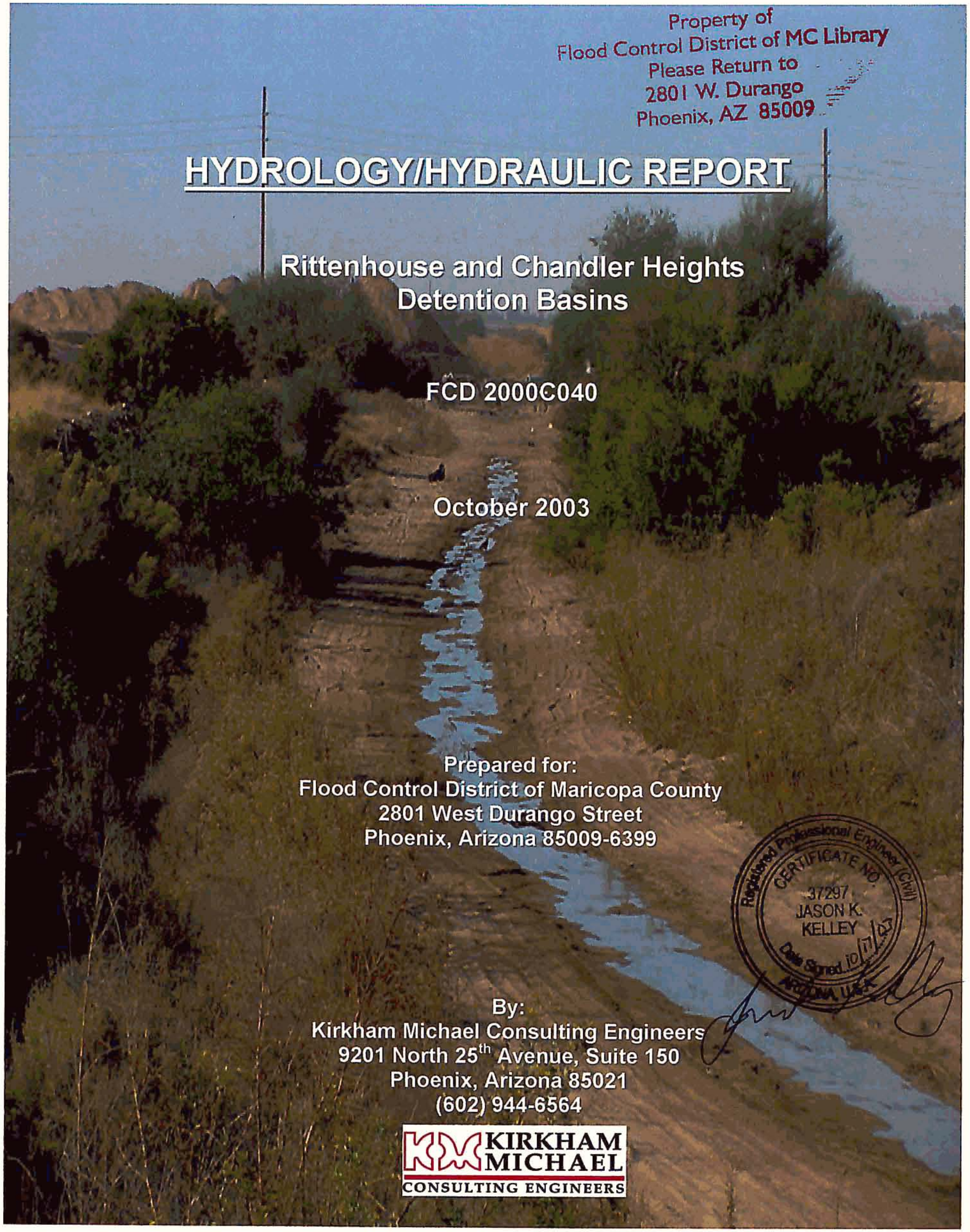

HYDROLOGY/HYDRAULIC REPORT

Rittenhouse and Chandler Heights Detention Basins

FCD 2000C040

October 2003

Prepared for:
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TABLE OF CONTENTS

Page

TABLE OF CONTENTS

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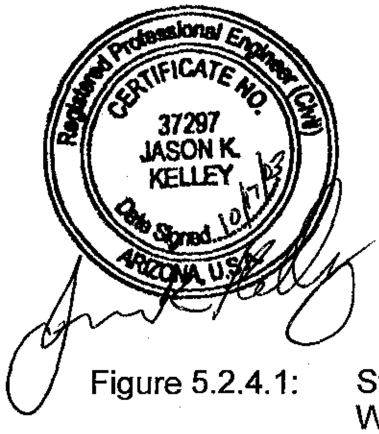


TABLE OF CONTENTS

Page

List of Figures

List of Tables

1.0 INTRODUCTION

In March 2002, the Flood Control District of Maricopa County (FCDMC) contracted with Kirkham Michael and Associates, Inc. (KM) for the Final Design of the Rittenhouse and Chandler Heights Detention Basins. This report presents a summary of the criteria and analyses that serve as the basis for the basin designs. Background information may be found in the Rittenhouse and Chandler Heights Predesign Reports. Detailed supporting documentation, calculations and analyses are also provided in the Rittenhouse and Chandler Heights Basin Design Data Report Calculations and Analysis Notebooks.

2.0 DESIGN CRITERIA

The design criteria for the Rittenhouse and Chandler Heights Basins include:

- Hydrology based upon the 100-yr, 24-hr future build-out conditions of the East Maricopa Floodway (EMF) watershed provided by the FCDMC that includes Capital Improvements Projects (CIPs) for flood control.
- Attenuation of the EMF peak flow downstream of the Chandler Heights Detention Basin to a maximum of approximately 6700 cfs.
- Drainage of detention basins within 36 hours after the cessation of the design storm duration (24 hours).
- Accommodation of features to provide opportunities for the use of the basins for recreation, recharge and other compatible multi-use purposes.

2.1 HYDROLOGY

The basin design hydrology is based upon the 100-yr, 24-hr future build-out conditions of the EMF watershed. The hydrology was provided by the FCDMC and includes proposed flood control CIPs in the EMF watershed.

2.2 EMF PEAK FLOW DISCHARGE ATTENUATION

The EMF attenuation criteria for this project is set at a maximum peak discharge of 6660 cfs immediately downstream of the Chandler Heights Basin. The predesign attenuation criteria along the EMF at Rittenhouse Road and at the County Line have been relaxed in belief that meeting the criteria at Chandler Heights will achieve the desired peak discharge attenuation and EMF freeboard. Attenuation criteria were established based upon a FCDMC assessment and evaluation of freeboard availability along the EMF.

2.3 BASIN DRAINAGE

The basins are designed to drain, as much as feasible, within 36 hours after the cessation of the 100-yr, 24-hr design storm.

2.4 MULTI-USE OPPORTUNITIES

The basins are designed to accommodate multi-use and aesthetics features and provide an opportunity for the basins to be used for recreation, recharge and other compatible multi-use purposes. Accommodations are made to the extent that they do not supplant the primary function and operation of the basins and channels for flood control.

2.5 OTHER DESIGN CRITERIA

The hydraulic analysis of the EMF is based upon a previous study (Collins-Pina/Tetra Tech, 2000). The study recommended changes to the channel for proposed multi-use and recreational improvements that would increase the channel n values (over the existing conditions) and include a low flow channel. These proposed improvements are part of the design criteria established by the FCDMC. Other design criteria were also developed during the process of design, established in meeting minutes or provided in the FCDMC drainage design guidelines and may be found in the Design Data Report.

3.0 HYDROLOGY

3.1 INTRODUCTION

The hydrology used for the basin designs include several hydrologic models and include both HEC-1 and HEC-RAS unsteady state flow simulation models. Generally, the HEC-1 models are used to analyze the EMF contributory watershed, route flow along the EMF and develop the input hydrographs for the HEC-RAS models. The HEC-RAS unsteady state flow models are used to analyze the lateral weirs, detention basins and the basin outlets.

3.2 HYDROLOGY SOFTWARE

HEC-1 and HEC-RAS are used to develop hydrology for the basin designs. Both were developed by the US Army Corps of Engineers, Hydrologic Engineering Center. For the design of the basins, HEC-1 Version 4.1 (June, 1998) and HEC-RAS Version 3.0.1 (March 2001) are used for the hydrologic analyses. Using different versions of the software models may produce results different than those presented in the supporting documentation.

TABLE 3.3 - Basin Design Models

Description	Model Type	Filename	Purpose
EMF-Watershed (NE Mesa)	HEC-1	WS1-NWM.DAT	Hydrologic analysis of the EMF watershed (future build-out conditions) Develops input hydrographs for Rittenhouse Basin HEC-RAS Model (RBD.PRJ)
EMF-Watershed (NW Mesa)	HEC-1	WS2-NEM.DAT	
EMF-Watershed (Queen Creek &	HEC-1	WS3-QCSW.DAT	
EMF-Watershed (SE Mesa)	HEC-1	WS4-SEM.DAT	
EMF-Watershed (Routing Model)	HEC-1	RT1-BASE.DAT	
Rittenhouse Basin	HEC-RAS	RBD.PRJ	Analysis of the Rittenhouse Basin and its impact on flow attenuation in the EMF. Develops input hydrograph for EMF HEC-1 (RCHB.DAT).
EMF – South of Rittenhouse Rd	HEC-1	RCHB.DAT	Routes flow in the EMF from the Rittenhouse Basin and provides input hydrographs for the Chandler Hts Basin HEC-RAS Model (CHBD.PRJ)
Chandler Heights Basin	HEC-RAS	CHBD.PRJ	Hydrologic/Hydraulic analysis of the Chandler Heights Basin & its impact flow attenuation of flow in the EMF. Develops the input hydrograph for the EMF– South of Chandler Heights Rd Model (SOFCH.DAT)
EMF – South of Chandler Heights Rd	HEC-1	SOFCH.DAT	A preliminary hydrologic evaluation of the EMF watershed below Chandler Heights Rd that includes the impact of the proposed Rittenhouse and Chandler Heights Detention Basins. The analysis is to be used by the FCDMC to evaluate freeboard in the EMF downstream of Chandler Heights Rd.

3.3 HYDROLOGIC MODELS

The hydrologic models used for the basin designs are identified in Table 3.3.

3.3.1 EMF Watershed Models

Several HEC-1 models are used to describe the hydrology for the EMF watershed upstream of the Rittenhouse Basin. The models were progressively developed in previous studies initiated by the FCDMC and were provided as the basis for design of the Rittenhouse and Chandler Heights Basins. The models describe the 100-year, 24-hour future build-out conditions for the EMF watershed and include proposed flood control CIPs.

TABLE 3.3.1 - Rittenhouse Basin Input Boundary Hydrographs

Source File: RT1-BASE.DAT Concentration Point	Destination File: RBD.PRJ	
	Channel	Cross Section
RWFLD1	EMF – Reach 4	17.082
RITTEN	Rittenhouse	820.00

All the "EMF Watershed Models" (Table 3.3) are necessary to develop the input boundary hydrographs for the HEC-RAS Unsteady State analysis of the Rittenhouse Basin. The models produce the input boundary hydrographs from model RT1-BASE.DAT needed for the HEC-RAS analysis of the Rittenhouse Basin (RBD.PRJ). Table 3.3.1 show the concentration point hydrographs and the corresponding cross section input boundary hydrographs.

3.3.2 Rittenhouse Basin Model (RBD.PRJ)

The Rittenhouse Basin HEC-RAS analysis is used to model the proposed Rittenhouse Detention Basin design and develop a hydrograph for the EMF downstream of the basin. The model includes a lateral weir between the EMF and the proposed basin and a flap gated outlet structure to drain the detention basin. The analysis uses input boundary hydrographs from RT1-BASE.DAT. From the results of the analysis, a hydrograph for EMF-Reach 4 Cross Section 16.000 (XS 16.000) can be obtained. This hydrograph represents the entire EMF watershed upstream of XS 16.000. The hydrograph is hard coded into a HEC-1 model (RCHB100.DAT) and routed to the Chandler Heights Basin.

3.3.3 EMF – South of Rittenhouse Road Model (RCHB.DAT)

This HEC-1 model is used to develop input boundary hydrographs for the Chandler Heights Basin HEC-RAS analysis. This model routes flow from the Rittenhouse Basin to the Chandler Heights Basin and includes the future build-out conditions hydrology for the Queen Creek and Sanokai Wash watersheds. Also included in this model is an update to a hard-coded hydrograph for the Sanokai Flood Retarding Structure on Queen Creek Wash.

The hydrograph from the Rittenhouse Basin HEC-RAS analysis (RBD.PRJ) at EMF-Reach 4, XS 16.000 is hard-coded into the model as concentration point RITBAS and routed in the EMF to the Chandler Heights Basin. The model will produce the input boundary hydrographs needed for the HEC-RAS analysis of the Chandler Heights Basin (CHBD.PRJ). The hydrographs for the concentration points and the corresponding cross section input boundary hydrographs are shown in Table 3.3.3.

TABLE 3.3.3 - Chandler Heights Basin Input Boundary Hydrographs

Source File: RCHBD100.DAT Concentration Point	Destination File: CHBD.PRJ	
	Channel	Cross Section
RQCS	EMF – Reach 3	13.084
CO508	QC/SW	5535
*(source is an initial run of CHBD.PRJ)	EMF – Reach 3	11.609

* See Section 3.3.4 for discussion of the source of the input boundary hydrograph at XS 11.609

3.3.4 Chandler Heights Basin Model (CHBD.PRJ)

The Chandler Heights Basin HEC-RAS analysis is used to model the proposed design of the Chandler Heights Detention Basin and develop a hydrograph for the EMF that meets the peak discharge design criteria of 6660 cfs downstream of the basin. The model includes a lateral weir, an emergency spillway, in-line weirs/drop structures and a gated outlet.

The model uses input boundary hydrographs for the EMF flow at Queen Creek Road (XS 13.084), the flow in Queen Creek/Sanokai Wash at the confluence of Queen Creek and Sanokai Wash (XS 4376.49) and the confluence of the EMF with Queen Creek/Sanokai Wash (XS 11.609) (Table 3.3.3).

Due to model instabilities, the junction option could not be used to represent the confluence of the EMF with Queen Creek/Sanokai Wash. Because there is no physical connection at this confluence, it is necessary to use a lateral inflow hydrograph at EMF XS 11.609. The hydrograph is created by an initial running of the to obtain a hydrograph at the downstream end of the Queen Creek (XS 1084.9) and then using the hydrograph as the lateral inflow hydrograph at EMF XS 11.609. The model is then re-run to accurately determine peak flow conditions in the EMF downstream of the confluence with Queen Creek/Sanokai Wash.

From the analysis, a hydrograph for EMF-Reach 3 XS 11.033 can be obtained which represents the entire EMF watershed upstream of XS 11.033..

4.0 HYDRAULICS

4.1 INTRODUCTION

The hydraulic analyses used to evaluate the operation of the detention basins was performed using HEC-RAS Unsteady State models. These models were used to establish the overall sizes, lengths and volumes of the detention basins, basin weirs and outlet structures. The detailed design of the weir and outlet structures are based upon separate analyses and will be provided in the Calculations and Analyses Notebooks.

For the Chandler Heights Basin, a steady state analysis was also conducted to design channel improvements, sedimentation basins and drop structures.

4.2 HYDRAULIC SOFTWARE

The HEC-RAS Version 3.0.1 (March 2001) hydraulic software developed by the US Army Corps of Engineers, Hydrologic Engineering Center (USACE-HEC) was used to design the basins, the hydraulic structures, and the new channel configurations. Version 3.0.1 is a release that includes a number of new features used in the design of the basins including Unsteady State Flow Analysis, Lateral Weirs, and Time Series Gate Openings.

4.2.1 HEC-RAS Software Bugs

HEC-RAS Version 3.0.1 also includes a number of software bugs. Among them is that at hydraulic connections such as gates or lateral weirs between a channel and a storage basin, hydrographs from the analyses may show some flow passing through the gate or over the weir even though at that time period the gates are shut or the water surface elevation is too low to pass over the weir. However, this does not significantly impact the results of the analyses.

4.2.2 Unsteady State Flow Analysis Instabilities

The addition of unsteady state flow analysis to HEC-RAS is a new, powerful tool that has the ability to route hydrographs through a network of channels, basins, weirs and other hydraulic structures. However, unsteady flow analysis is more complex and can be extremely difficult compared with steady flow analysis because of model instabilities. Instabilities result from the program having difficulty converging on a solution. Even minor changes to input parameters can dramatically affect the stability of a model.

Model instabilities occur for many reasons. According to the HEC-RAS User's Manual, instabilities can occur at low flows because:

1) Flow depths are small. As flow increases between time steps, flow depth can increase dramatically. If flow depth increases significantly between time steps, oscillations can occur in the analysis and can grow to the point at which the solution becomes unstable.

2) At low flows or shallow depths, water is more likely to be flowing in a pool or riffle sequence. At the riffles, the flow may be passing

through critical depth and going supercritical. The current version of the unsteady flow solver in HEC-RAS cannot handle supercritical flow or even flows approaching critical depth. Such conditions may cause may cause the model to go unstable.

Instabilities can also occur when analyzing inflow/outflow between two hydraulically connected features, such as flow between a basin and a channel through a gated opening or spillway/weir. Typically, instabilities occur when the basin and the channel water surface elevations are very close and the hydraulic connection (gate, spillway or weir) is in operation. Under these conditions, reiteratively solving for the outflow/inflow through the hydraulic connections combined with the resulting fluctuation in water surface elevations between the basin and the channel makes it difficult to converge on a solution.

4.2.3 Improving Model Stability

There are ways to help prevent model instabilities without affecting the results. HEC-RAS has a pilot channel option to help prevent low flow instabilities. The pilot channel does not physically exist but is used theoretically during the analysis of low flows. At higher flows, the pilot channel is ignored.

Another method to provide stability during periods of low flow that occur at the onset of a storm event is to increase the initial flow in the Initial Conditions of the Unsteady Flow Data Editor. The initial flow is used to perform a backwater analysis to compute stages at each cross section. By increasing the initial flow, supercritical flow depths can be avoided at the start of the analysis and instabilities can be avoided. The effect increase diminishes quickly as the model re-establishes the normal water surface elevation at each cross section.

Instabilities that arise from the analysis of hydraulically connected features can be resolved by modifying the model to avoid the calculation of flow between the features when they are at similar water surface elevations. This requires more familiarity with the model to insure the model continues to reflect overall operating conditions and that the results are not compromised. In the case of a gate opening, this can be done by opening the gate earlier, later, or only during periods where there is significant difference between the water surface elevations of the hydraulic features. For a weir or spillway, minor changes in the weir/spillway elevation or length and changes in the channel or basin configuration can improve model stability by increasing the difference in water surface elevations and/or the frequency at which calculations are made at similar water surface elevations.

Another way to improve model stability is to modify Computation Options and Tolerances in the Unsteady State Flow editor or by modifying computation intervals. Modifying these options may impact the analysis results. According the HEC-RAS User's Manual, increasing the default calculation tolerances can result in computational errors in the water surface profile.

4.3 HEC-RAS MODELING OF FLAP GATE OUTLETS

HEC-RAS Version 3.0.1 does not specifically model flap gate outlets, therefore, during predesign, a process using manual calculations and an iterative procedure of balancing water surface elevations, gate opening heights and flow discharge was used to model flap gates. Subsequent to the predesign analysis, another procedure was developed that produced results comparable to the predesign procedure, but was more simple to implement.

The new procedure uses the HEC-RAS Time Series Gate Openings Option in the Unsteady State Flow Data Editor to model flap gates. The procedure involves "homing in" on the time at which flap gates would open and then opening the gates. When the detention basin water surface elevation is above the channel water surface elevation, the flap gates are assumed to open and discharge flow into the channel, otherwise, the gate remains closed. A typical procedure to model a flap gate outlet is as follows:

1) Either assume a time when the flap gates will be open or let the gates remain closed for the entire run and set the Time-Series Gate Openings in the Unsteady Flow Data Editor accordingly.

2) Run the model and estimate from the basin outlet stage hydrograph the time at which the basin water surface elevation will exceed the channel water surface elevation.

3) Revise the Time-Series Gate Openings in the Unsteady Flow Data Editor so that the gates open at (or slightly after) that point in time and rerun the analysis.

4) Review the basin outlet stage hydrograph. If the Time Series Gate Opening data agrees with the time at which the basin water surface elevation begins to exceed the channel water surface elevation, the analysis is complete. If the times are significantly different, adjust the Time Series Gate Opening data to agree with the basin outlet stage hydrograph and repeat Step 4.

If the analysis becomes unstable due to the gate opening, adjust the time so that the gate does not open until the difference between the basin water surface elevation and the channel water surface elevation is larger. If the instability occurs later in the analysis (as the basins is draining) and the basin water surface elevation begins to approach the channel water surface elevation, it might be appropriate and necessary to close the gate.

4.4 HYDRAULIC MODELS

The HEC-RAS models used for the basin designs were generally described previously in the Hydrologic Model Section of this report. This section discusses in more detail the HEC-RAS analyses for the basins.

4.4.1 Rittenhouse Basin Analysis

This HEC-RAS analysis is used to model the design of the Rittenhouse Detention Basin and develop a hydrograph for the EMF downstream of the basin. The model includes a lateral weir between the EMF and the proposed basin and a flap gated outlet structure to drain the detention basin. The analysis uses input boundary hydrographs from RT1-BASE.DAT. From the HEC-1 analysis, a hydrograph for EMF-Reach 4 Cross Section 16.000 can be obtained. This hydrograph represents the entire EMF watershed upstream of Cross Section 16.000. The hydrograph is hard coded into a HEC-1 model (RCHB100.DAT) and routed to the Chandler Heights Basin.

It is not believed debris accumulation at the EMF bridge crossings at Rittenhouse Road have been problematic, however, to evaluate possible impact of debris accumulation on the Rittenhouse Basin design analysis, the bridge pier widths for the Rittenhouse Road bridge and the SPRR bridge were increased four-fold and the design analysis rerun. The results indicate that such debris accumulation would have no significant effect and therefore the original bridge sections were used unchanged for the EMF analysis.

<u>4.4.1.1 Geometric Data</u>

Cross Sections and Bridge Sections

The EMF cross sections, bridge sections and 'n' values remain unchanged from the HEC-RAS model provided by the FCDMC (Collins-Pina/Tetra Tech, 2000). The study model incorporated changes to the channel for proposed future multi-use and recreational improvements that increased channel 'n' values (over the existing conditions) and included a meandering low flow channel, approximately eight feet wide by two to three feet deep.

For the Rittenhouse Channel, two channel cross sections and a junction at the confluence were added to the geometric data.

Lateral Weir and Gated Outlets

Beginning at EMF XS 16.940, a lateral weir between the EMF and the proposed Rittenhouse Basin is included in the geometric data. At the end of the lateral weir, a flap-gated outlet is also modeled. The lateral weir is analyzed as a broad-crested weir with a weir coefficient of 2.3 based upon a detailed investigation and estimation of weir coefficient performed during the Predesign phase (Predesign Reports).

Detention Basin

The stage-volume curve is based upon a basin that accommodates landscaping and aesthetic features to enhance the basin appearance.

4.4.1.2 Unsteady Flow Data

Boundary Conditions

EMF - Reach 4 RS 17.082
This is the upstream end of the EMF in the analysis. A hydrograph obtained from the EMF Watershed Models for the upstream watershed is input as a boundary condition (Section 3.3.1).

EMF - Reach 4 RS 16.93 LW
This is the lateral weir and flap-gate outlet between the EMF and the Rittenhouse Detention Basin. A boundary condition is necessary at this location to model the outlet. Time Series Gate Opening Data is used as the boundary condition. In the data, the flap-gates are completely opened approximately at the time the basin water surface elevation exceeds the EMF water surface elevation. The HEC-RAS model then automatically calculates the discharge through the flap gates from the difference in water surface elevations between the basin and outlet channel (Section 4.3).

EMF - Reach 4B RS 16.00
This is the downstream end of the EMF in the analysis. Normal depth calculations using the approximate EMF channel slope (0.00031 ft/ft) is used as the downstream boundary condition.

Rittenhouse Channel - Main Channel RS 820.0
This is the upstream end of the Rittenhouse Channel in the hydraulic analysis. A flow hydrograph developed from the EMF Watershed Models representing the watershed upstream of this location is input as the boundary condition (Section 3.3.1).

Initial Conditions

Initial flow conditions are required at three locations and an initial elevation is required for the Rittenhouse Basin Table 4.4.1.2). Initial flow conditions are based upon the flow rate of at each cross section at the beginning of the analysis unless the flow rate is zero, at which a nominal flow rate of 2 cfs is entered to avoid model instability. The initial elevation is set at the bottom of the detention basin.

EMF - Reach 4 RS 16.93 LW
This location is at the lateral weir and flap-gate outlet between the EMF and the Rittenhouse Detention Basin. To model the basin flap-gate outlet, Time Series Gate Opening Data is used as the boundary condition. In the data, the gates are opened when the basin water surface elevation exceeds the EMF water surface elevation.

EMF - Reach 4B RS 16.00
This location is at the downstream end of the EMF in the analysis. Normal depth calculations using the approximate EMF channel slope (0.00031 ft/ft) is used as the downstream boundary condition.

Rittenhouse Channel - Main Channel RS 820.0
This location is at the upstream end of the Rittenhouse Channel. A flow hydrograph obtained from the EMF Watershed Models for the Rittenhouse contributory watershed upstream of this location is input as the boundary condition (Section 3.3.1).

Initial Conditions

Initial flow conditions are required at three locations and an initial elevation is required for the Rittenhouse Basin. Initial flow conditions are based upon the flow rate of at each cross section at the beginning of the analysis unless the flow rate is zero, at which a nominal flow rate of 2 cfs is entered to avoid model instability. The initial elevation for the Rittenhouse Basin is set at the bottom of the detention basin. Input initial flow conditions and elevations are shown in Table 4.4.1.2.

TABLE 4.4.1.2 - Rittenhouse Initial Flow & Elevation Conditions

Location	Initial Flow (cfs)	Initial Elevation (ft)
EMF Reach 4, RS 17.082	75	-
EMF Reach 4B, RS 16.251	77	-
Ritt Channel, Main Channel RS 820.00	2	-
Rittenhouse Basin	-	1311

4.4.2 Chandler Heights Basin Analyses

Two separate HEC-RAS models are used to model the proposed Chandler Heights Basin design.

An unsteady state model (CHBD.PRJ) is used to size the proposed basin and associated structures. The model contains several structures including a lateral weir, an emergency spillway and a flap-gated outlet.

The analysis uses input boundary hydrographs from RCHB100.DAT for flow in the EMF at Queen Creek Road and in Queen Creek after the confluence with Sanokai Wash. The analysis provides a hydrograph representing the EMF watershed upstream of the Chandler Heights Road that includes the proposed Rittenhouse and Chandler Heights Basins (EMF Reach 3 RS 11.033).

A steady state model (QCSW.PRJ) is used for the analysis and design of channel improvements and drop structures in Queen Creek and Sanokai Wash. It includes the proposed sedimentation basin in Queen Creek, channel drop structures and inlet and outlet structures to the sedimentation basins.

It is not believed debris accumulation at the EMF bridge crossing at Chandler Heights has been problematic, however, to evaluate the impact of debris accumulation at the bridge, pier widths for the Chandler Heights Road bridge were increased four-fold and the design analysis rerun. The results indicate that such debris accumulation would have no significant effect and therefore the original bridge sections were used unchanged for the EMF analysis

4.4.2.1 Geometric Data

Cross Sections and Bridge Sections

Most of the EMF cross sections and bridge sections remain unchanged from the HEC-RAS model provided by the FCDMC. These cross sections and 'n' values are based upon a previous study conducted by Collins-Pina/Tetra Tech (Collins-Pina/Tetra Tech, 2000). The model incorporated proposed changes to the channel for future multi-use and recreational improvements that increased channel 'n' values (over the existing conditions) and included a meandering low flow channel, approximately eight feet wide by two to three feet deep.

The design model contains EMF cross sections that were modified to reflect the proposed relocation of the existing drop structure to just upstream of the Chandler Heights Bridge (previously at ~XS 11.321) to upstream of the proposed basin outlet (~XS 11.794).

The geometric data for Queen Creek and Sanokai Wash is based upon the realignment and channelization of the existing washes adjacent to the proposed detention basin. The steady state model geometry, used to design channel improvements and drop structures, extends to Higley Road along both Queen Creek and Sanokai Wash. To maintain model stability, the geometry in the unsteady state model extends only to the upstream end of the proposed lateral weir (downstream of the confluence of Queen Creek and Sanokai Wash).

Lateral Weirs and Gated Outlets

A lateral weir is located at Queen Creek ~XS 5377 that connects the Queen Creek channel to the Chandler Heights Basin. The structure is analyzed with a weir coefficient of 2.44 based upon an estimation of weir coefficients performed during the predesign phase of the project.

Between the EMF and the proposed basin, two lateral weir structures are included in the geometry data. One models an emergency spillway of the basin into the EMF (~XS 11.988). The spillway provides emergency relief if the basin stage exceeds the 100-year stage (approximately 1306.5). The other lateral weir contains gates and models the flap-gated outlet used to drain the basin (~XS 11.741).

Lateral weirs and outlet geometry is not included in the steady state model as it is not needed to analyze the channel and drop structures.

Detention Basin

The stage-volume curve is based upon a footprint that accommodates landscaping and aesthetic features to enhance the basin appearance. There is no detention basin geometry in the steady state model.

4.4.2.2 Flow Data

Boundary Conditions – Unsteady State Model

EMF - Reach 3 RS 13.084
This location is at the upstream end of the EMF in the model (just downstream of Queen Creek Rd). A flow hydrograph (RQCS) representing the EMF watershed upstream of this location is input as the boundary condition. The hydrograph is developed in an HEC-1 model (RCHB100.DAT) that routes flow from the EMF downstream of the Rittenhouse Basin to the Chandler Heights Basin (Section 3.3.1).

EMF - Reach 3 RS 11.741 LW

This location is at the flap-gated outlet between the EMF and the Chandler Heights Basin. To model the flap-gate outlet, Time Series Gate Opening Data is used as the boundary condition. In the data, the flap-gates are completely opened when the basin water surface elevation exceeds the EMF water surface elevation.

EMF - Reach 3 RS 11.609

This location is at the confluence of the EMF and the Queen Creek channel. Because of model instabilities, there is no "physical" connection of Queen Creek and the EMF in the HEC-RAS model. Instead, the confluence is modeled by adding a Lateral Inflow Hydrograph as a boundary condition at this location in the EMF. It is important to note that because the confluence has no "physical" connection in the model, it is necessary to perform an initial run of the Chandler Heights Basin HEC-RAS model in order to obtain the Lateral Inflow Hydrograph at ~XS 11.609 (Section 3.3.4).

EMF - Reach 3 RS 11.033

This location is at the downstream end of the EMF in the analysis. Normal depth calculations using the approximate EMF channel slope (0.0003 ft/ft) is used as the downstream boundary condition.

Queen Creek - R1 RS 1084.9

This location is at the downstream end of Queen Creek just prior to the confluence with the EMF. As the downstream boundary condition, normal depth calculations are used with a friction slope of 0.01 ft/ft.

Queen Creek - R1 RS 5535

This location is just upstream of the lateral weir and downstream of the confluence of Queen Creek and Sanokai Wash channels. A flow hydrograph representing the respective contributory watersheds of Queen Creek and Sanokai Wash upstream of this location is input as the boundary condition. The hydrograph (CO508) is obtained from the

TABLE 4.4.2.2.1 - Chandler Heights Basin Initial Flow & Elevation Conditions

Location	Initial Flow (cfs)	Initial Elevation (ft)
EMF Reach 3, RS 13.084	700	-
Queen Creek R1, 5535	400	-
Chandler Heights Basin	-	1296

Table 4.4.2.2.2 Steady State Model Starting Water Surface Elevations

River	Reach	Upstream	Downstream
Sanokai Wash	R1	Normal Depth S = 0.002 ft/ft	QC/SW Junction
Queen Creek	R1	Normal Depth S = 0.0003 ft/ft	QC/SW Junction
Queen Creek	R2	QC/SW Junction	EMF/QCSW Junction
EMF	Reach 3	Normal Depth S = 0.0003 ft/ft	EMF/QCSW Junction
EMF	Reach 3-Lower	EMF/QCSW Junction	Normal Depth S = 0.0003 ft/ft

HEC-1 model RCHB100.DAT which develops the hydrology for the Queen Creek Wash and Sanokai Wash watersheds (Section 3.3.1).

Initial Conditions—Unsteady State

Initial flow conditions are required at the upstream reach locations in the unsteady state model along with an initial basin elevation for the Chandler Heights Basin. Initial flow conditions and initial basin elevation are shown in Table 4.4.2.2.1.

For model stability, the initial flow rates used in the analysis are larger than actual input hydrograph information at the initial time period at each location (Section 4.2.3). The increase does not adversely impact the analysis results.

Starting Water Surface Elevations and Flow Data– Steady State Model

Starting Water Surface Elevations
Starting water surface elevations for the Chandler Heights Basin steady state analysis are provided in Table 4.4.2.2.2.

Flow Data
Flow data for the Chandler Heights Basin steady state analysis is derived from HEC-1 models and the Unsteady State Analysis of the Chandler Heights Basin (CHB.PRJ). Flow data and their sources are identified in Table 4.4.2.2.3.

Table 4.4.2.2.3 Steady State Model Peak Flow Changes

River	Reach	Cross Section	Peak Discharge (cfs)	Source Hydrograph
Sanokai Wash	R1	2016.8	3310	RCHB100.DAT (CO508A)
Queen Creek	R1	10723	2930	RCHB100.DAT (CO484)
Queen Creek	R2	6135	5536	RCHB100.DAT (CO508)
Queen Creek	R2	5382	5536	CHB.PRJ (5382)
Queen Creek	R2	5157	4872	CHB.PRJ (5157)
Queen Creek	R2	4932	4010	CHB.PRJ (4932)
Queen Creek	R2	4707	3090	CHB.PRJ (4707)
Queen Creek	R2	4482	2312	CHB.PRJ (4482)
EMF	Reach 3	13.084	3859	RCHB100.DAT (RQCS)
EMF	Reach 3	11.741	3804	CHB.PRJ (11.741)
EMF	Reach 3	11.609	4357	CHB.PRJ (11.609)
EMF	Reach 3-Lower	11.572	6627	CHB.PRJ (11.572)

5.0 SUMMARY OF BASIN DESIGNS AND ANALYSES RESULTS

5.1 INTRODUCTION

The analysis and design of the detention basins were developed through an intensive evaluation and refinement of alternative basin designs and configurations during the Pre-design phase and subsequent design phases. This section briefly discusses the process of optimizing basin designs conducted during the 30% design phase and also presents a summary of the basin designs and related analytical results for the detention basins.

5.2 RITTENHOUSE BASIN

5.2.1 Optimizing Basin Design

During the 30% Design Phase, the initial Predesign recommendations were further developed for changes in design criteria, refinement for multi-use opportunities, and more detailed design of the structures. Optimizing the basin design consisted of using a revised basin volume that included additional area for landscaping and then analyzing various lateral weir lengths, elevations and outlet sizes. Based upon the weir optimization analysis, the weir alternative that achieved the best combination of flow attenuation, weir length and weir crest elevation was then selected as the configuration for the design weir.

Weir length and elevation both have a direct impact on effectiveness of the basin to attenuate flow in the EMF, therefore, a series of analyses were performed at various weir lengths and elevations. The analyses indicated that each weir length had an elevation at which the weir was most effective.

Sizing the basin outlets was simplified since the outlet did not have an impact on the EMF peak flow rate. The outlet was sized to drain the basin within the 36 hours after the storm event. While the basin cannot be drained completely within 36 hours due to flow in the EMF, it will drain to within a few inches.

5.2.2 Value Engineering

After the 30% design review, a Value Engineering (VE) session was conducted on the detention basin designs. The recommendations from the VE session, along with other subsequent design review comments, were accommodated in the development of the basin design plans. VE recommendations and other revisions made to address subsequent design review comments were not necessarily based upon the optimal operation of the basin. Therefore a comprehensive optimization process, as performed for the 30% design phase, was not performed. The basin design, however, should still reflect a relatively efficient basin configuration.

5.2.3 Basin Design

<u>5.2.3.1 Detention Basin</u>

The proposed Rittenhouse Detention Basin is approximately 130 acres in size (~158 acres with landscape area, ~172 acres with area south of the Pecos Rd. alignment) and has an estimated stage-storage volume relationship as shown in Table 5.2.3.1. The basin bottom elevation is 1311 ft, the weir elevation is set at 1315 ft (sloping to 1314.75 ft over the ~800-ft weir length), and the minimum top of basin elevation is approximately 1319.5 ft.

TABLE 5.2.3.1 - Rittenhouse Basin Stage vs. Storage Volume

Basin Elevation (ft)	Area (acres)	Cumulative Storage Volume (acre-ft)
1311	0	0
1312	44	22
1313	109	99
1314	118	213
1315	119	331
1316	120	451
1317	121	571
1318	122	693

*Peak basin WSEL occurs at ~1316.67 for a maximum basin storage volume of ~530 acre-ft

In the future, the basin footprint should not be significantly modified and fill should not be imported into the basin without investigating the impact a reduction in the basin storage volume will have on the basin and the EMF drainage system, including the proposed Chandler Heights Basin. However, it is felt that the bottom can be regraded to provide additional relief as long as positive drainage of the basin can be achieved.

5.2.3.2 Lateral Weir/Basin Outlet

The proposed lateral weir is ~ 800 ft in length and varies in width for aesthetic purposes, however, a minimum 15 ft width across the top is provided for vehicular access. At the upstream end of the weir the weir elevation is set at 1315.00. The weir elevation gradually decreases in elevation at approximately the same slope of the EMF (~0.0003 ft/ft) to 1314.75 at the end of the weir. The basin outlet is a 3–6' x 4' flap-gate outlet built into the lateral weir.

5.2.4 Analysis Results

The results of the Rittenhouse Basin design analysis are described in the hydrographs presented in Figures 5.2.4.1 – 5.2.4.3.

Figure 5.2.4.1 shows the stage hydrograph for the Rittenhouse Basin and the EMF channel upstream and downstream of the lateral weir. The results show the basin elevation peaking at ~1316.7 for a peak storage volume of ~535 acre-ft. At this elevation, almost 3 feet of freeboard is provided around the perimeter of the basin (minimum top basin elevation is 1319.50). The figure also shows the basin drains to ~1311.25 within 36 hours after the storm event, leaving 0.25 ft to drain from the basin through percolation. The presence of protracted flow in the EMF after the storm event prevents complete basin drainage through the outlet within 36 hours, however, the remaining water should percolate quickly. Figure 5.2.4.1 also identifies the time at which the

Figure 5.2.4.1 - Stage Hydrographs at the EMF/Rittenhouse Basin Weir and Flap Gate Outlet

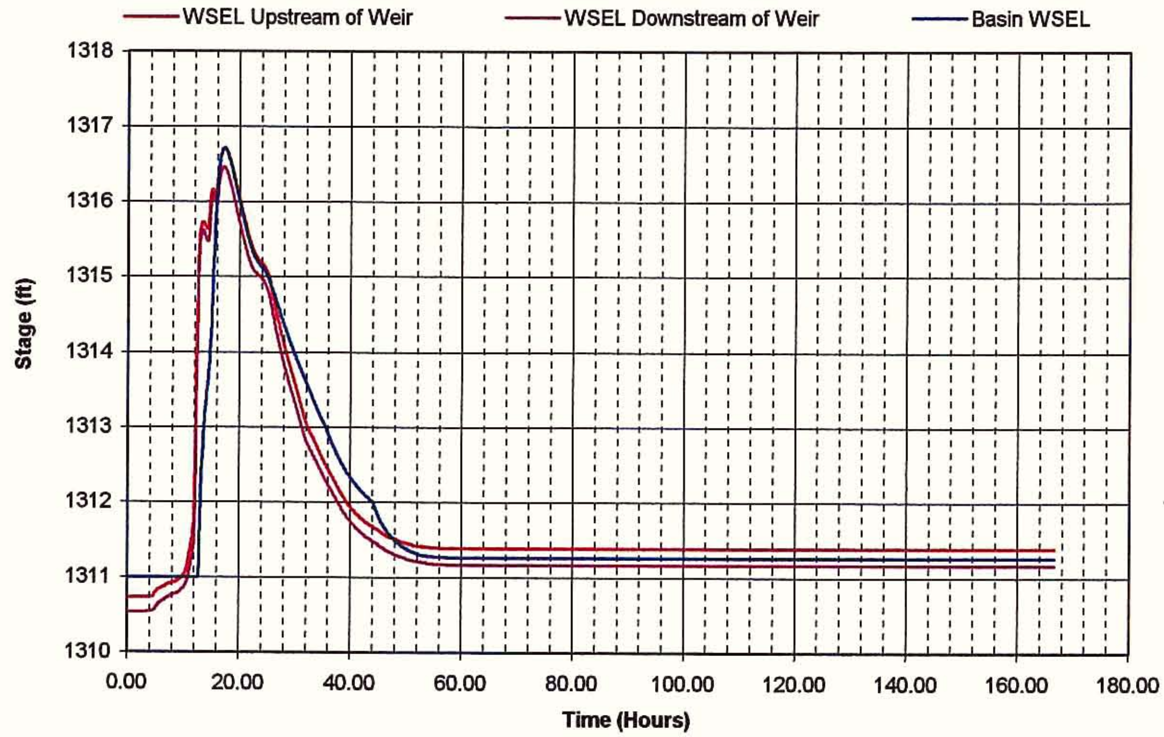


Figure 5.2.4.2 - Flow Hydrographs at the EMF/Rittenhouse Basin Weir and Flap Gate Outlet

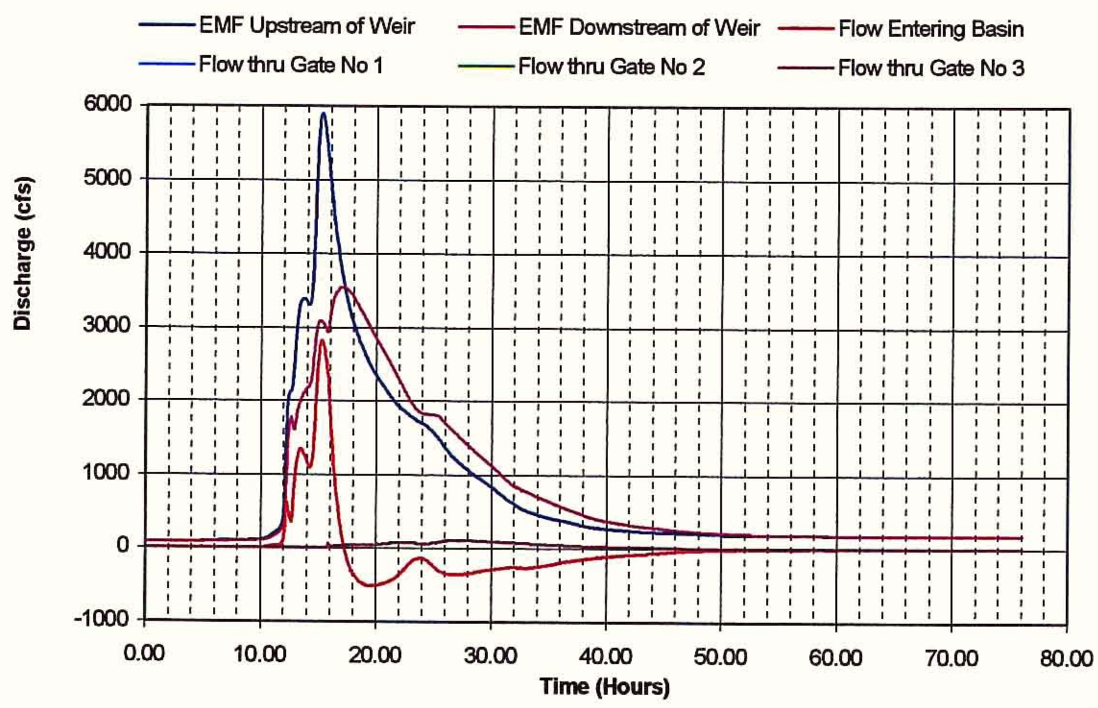
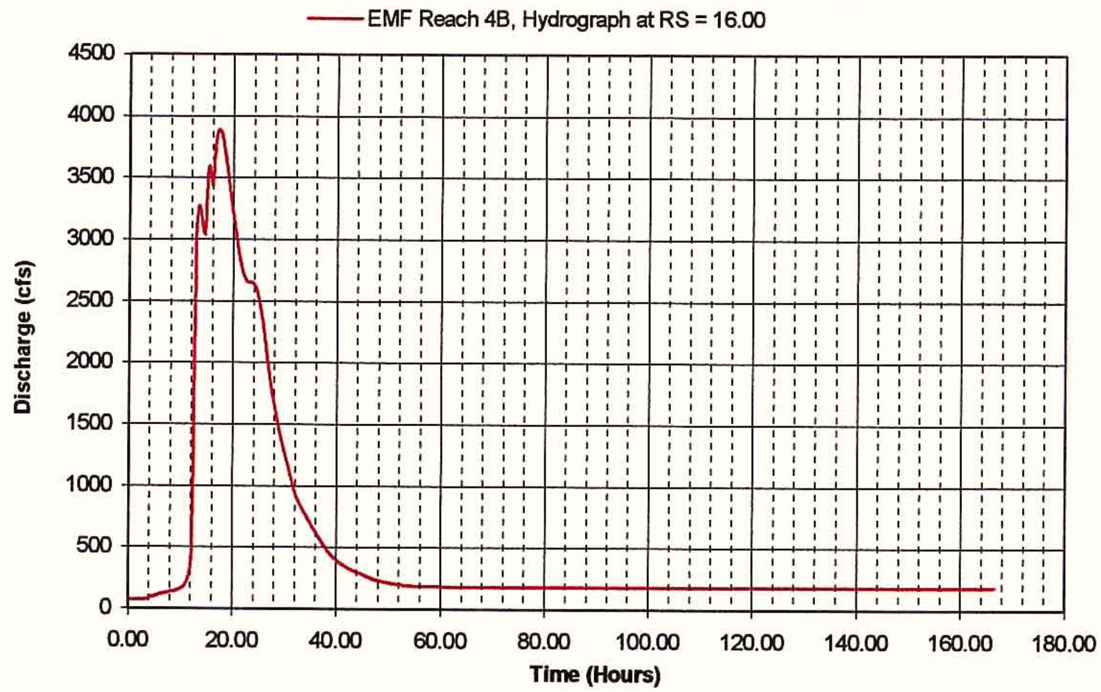


Figure 5.2.4.3 - Flow Hydrograph
Downstream of Rittenhouse Basin
(Exported to HEC-1 Model RCHBD60.DAT)



flap gates should start to open. At 15:50 hours into the storm event, the basin water surface elevation (WSEL) starts to exceed the WSEL in the EMF at the flap gate outlet ("WSEL Downstream of Weir").

Figure 5.2.4.2 shows the impact of the operation of the basin and lateral weir/ flap gate outlet on flow in the EMF channel.

The peak flow upstream of the weir of ~5900 cfs is reduced to ~3080 cfs after the weir. The amount of flow passing over the lateral weir ("Flow Leaving EMF") and into the detention basin peaks around 2820 cfs. After the peak, flow into the basin quickly drops and flow begins leaving the basin and flowing back over the lateral weir and through the flap gate outlet back into the EMF (a negative value for "Flow Leaving the EMF" means flow is entering the EMF).

Figure 5.2.4.3 shows the resulting EMF hydrograph downstream of the Rittenhouse Basin and downstream of the Rittenhouse Channel at EMF-Reach 4B, XS 16.000. The results show the peak flow in the EMF, downstream of the Rittenhouse Channel, has been attenuated to ~3890 cfs. This hydrograph is routed down the EMF for use in the analysis of the Chandler Heights Basin.

5.3 CHANDLER HEIGHTS BASIN

5.3.1 Optimizing Basin Design

Various design goals were accounted for in the optimization of the Chandler Heights Basin. These included accommodations for future multi-use activities within the EMF and basin, detailed design and analysis of the Queen Creek and Sanokai Wash channels, sedimentation basin, channel drop structures, lateral weir structure and detention basin outlet structure.

The structures and channels integrated into the Chandler Heights Basin were evaluated to assess the impact that each had on the entire system. Features, such as the channel drop structures, were also analyzed individually in order to design for a range of conditions not assessed in the overall evaluation.

Optimization of the Chandler Heights Basin system consisted of:

- Adjusting the overall design of the Chandler Heights Basin system based upon the design of the Rittenhouse Basin;
- Reconfiguring the basin stage/volume to accommodate landscaping features while minimizing basin excavation and maximizing basin storage effectiveness;
- Developing a lateral weir length/elevation and Queen Creek channel configuration to:
  - maximize the attenuation of EMF flow;
  - maintain acceptable channel freeboard;
  - maintain acceptable channel velocities.
- Optimizing the size of the basin outlet structure to:
  - minimize basin storage requirements;
  - efficient attenuation of EMF flow;
  - drain the basin within 36 hours after the 24-hour storm event.

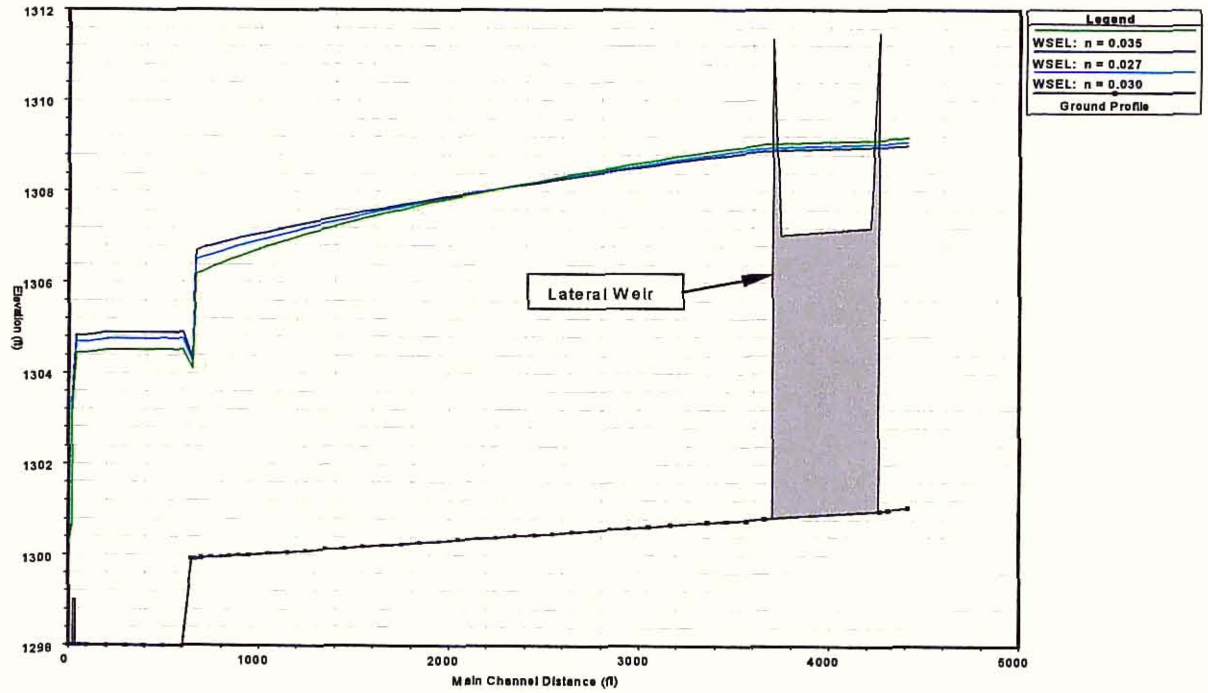
As a result of the optimization of the basin design:

- the basin outlet consists of a 2-6'x4' RCBC with flap gates;
- the lateral weir is 800 ft in length with an average height above the Queen Creek channel of 6.25 feet;
- the proposed basin contains 1350 acre-feet of volume during the 100-year event while maintaining a minimum of 2 ft. of freeboard;
- the Queen Creek channel downstream of the lateral weir structure has a bottom width of 50 feet.

5.3.2 'n' Value Sensitivity Analysis

At the 30% and 60% design phases, sensitivity analyses were performed using a range of n-values on both Queen Creek and the EMF to evaluate the impact

Figure 5.3.2.1: Queen Creek Channel 'n' Value Sensitivity Analysis



that variations in Manning's 'n' would have on the function of the proposed Chandler Heights Basin, the EMF and the Queen Creek channel

Figure 5.3.2.1 shows the impact 'n' values of 0.027, 0.030, and 0.035 have on the water surface elevation in the Queen Creek channel. At the critical location of the lateral weir, the increase in 'n' value from 0.027 to 0.035 increases the water surface profile by approximately 0.1 feet. The increase in water surface elevation increases the amount of flow over the lateral weir 100-200 cfs (Figure 5.3.2.2). This increases the maximum stage in the detention basin by 0.5 to 1.0 feet and the amount of detention storage by 100 to 270 acre-ft. Channel velocities in Queen Creek decrease by 0.5 fps or less.

Due to the response of the detention basin stage and storage resulting from changes in 'n' value, the design 'n' value should account for future vegetative growth and the channel should be maintained to insure the proper operation of the channel and the Chandler Heights Detention Basin. Based upon the results, an 'n' value selected for use in design based on FCDMC criteria was 0.030 (Table 6.11, Drainage Design Manual for Maricopa County, Volume II, Hydraulics, January 28, 1996). This value accounts for significant growth in the channel, including grass and shrubs.

The effects of 'n' value variation were also evaluated in the EMF. Figure 5.3.2.3 shows the EMF water surface profiles corresponding to 'n' values of 0.030, 0.035, and 0.040. The water surface varies by approximately 0.5 feet

Figure 5.3.2.2 - Flow Across Weir into Chandler Heights Basin

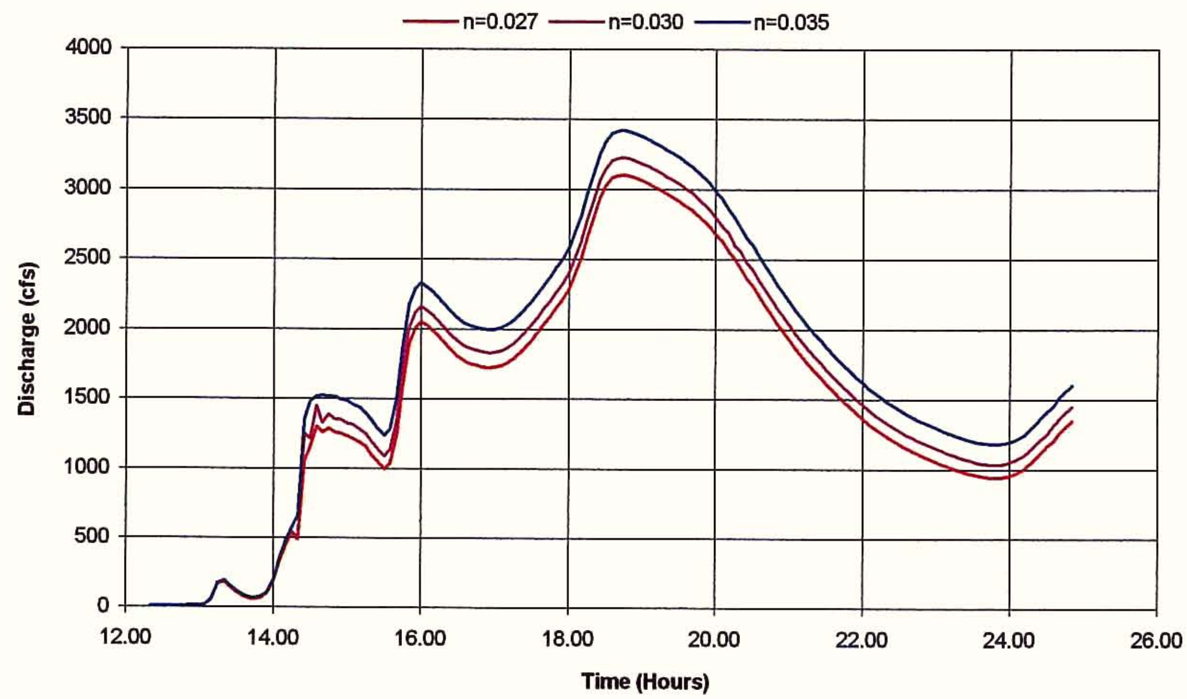
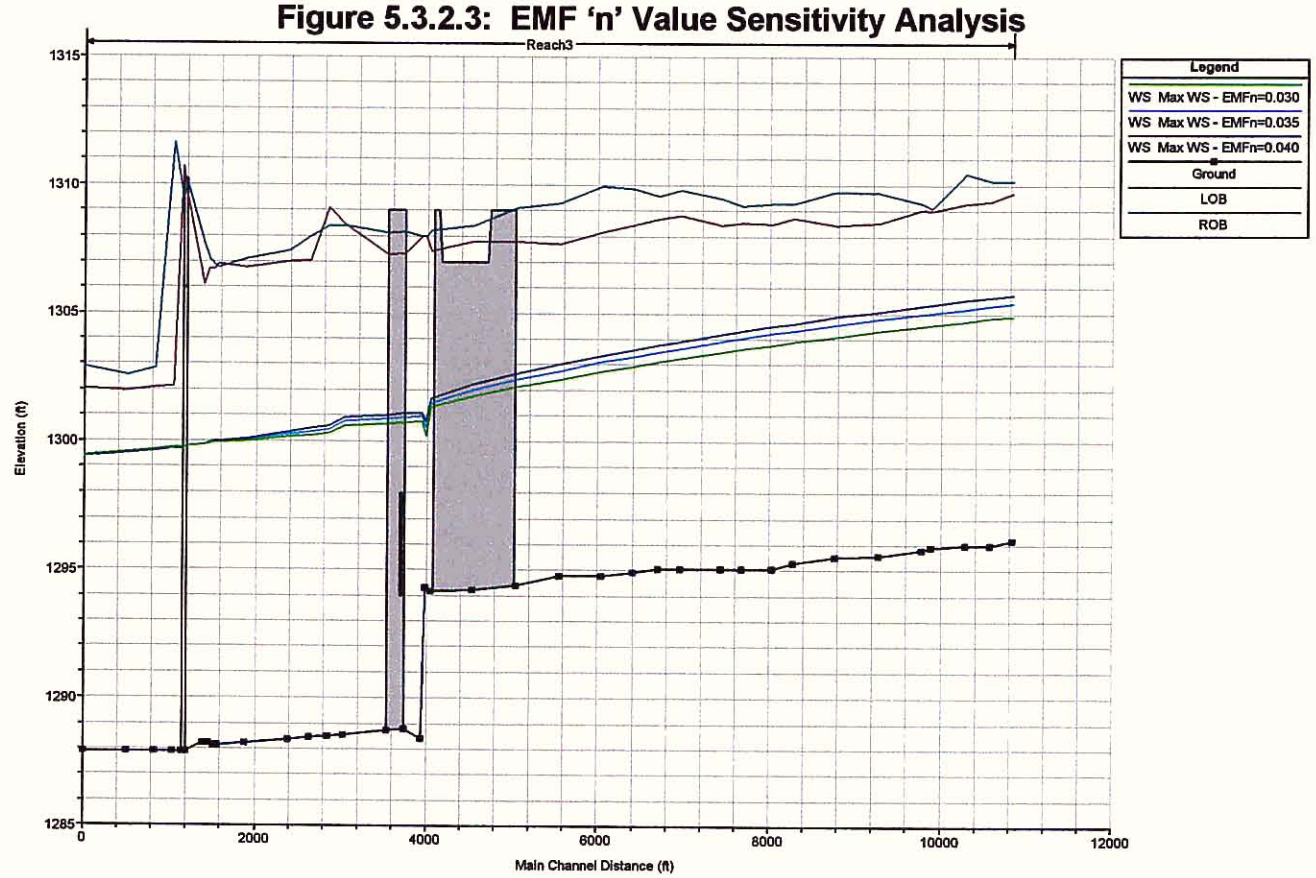


Figure 5.3.2.3: EMF 'n' Value Sensitivity Analysis



between profiles and the velocity varies by less than 0.5 fps along the entire reach. The slight increase in the EMF water profile slightly increases the tailwater on the basin's flap-gate outlet. However, the impact is insignificant, increasing the basin stage by 0.1 ft and the basin volume by approximately 10 acre-ft. Ultimately, an n-value of 0.040 was used for design in accordance to a previous study and at the direction of the FCDMC (Section 4.4.2.1).

5.3.3 Basin Design

5.3.3.1 Detention Basin

The Chandler Heights Detention Basin is approximately 230 acres in area (including landscape areas) and has an estimated stage-storage volume relationship as shown in Table 5.3.3.1. The basin bottom elevation is at 1296 and the minimum top basin elevation is 1309. This allows for over two feet of freeboard at the peak basin water surface elevation (~1306.5) during the 100-yr, 24-hr event. Due to existing topography, the northern end of the basin is deeper and has significantly more freeboard than the southern end.

In the future, the basin footprint should not be significantly modified and fill should not be imported into the basin without investigating the impact a reduction in the basin storage volume will have on the basin and the EMF. However, it is felt that the bottom can be regraded to provide additional relief as long as positive drainage of the basin can be achieved.

TABLE 5.3.3.1 - Chandler Heights Basin Stage vs. Storage Volume

Basin Elevation (ft)	Area (acres)	Cumulative Storage Volume (acre-ft)
1296	0	0
1297	19	9
1298	53	45
1299	58	101
1300	125	192
1301	174	342
1302	175	516
1303	177	693
1304	179	871
1305	181	1050
1306	182	1232
1307	184	1415
1308	186	1600
1309	187	1786

*Peak basin WSEL occurs at ~1306.5 for a maximum basin storage volume of ~1325 acre-ft

5.3.3.2 Lateral Weir

The proposed concrete lateral weir is a 800 feet long. The width of the weir varies for aesthetic purposes but the proposed minimum width is 15 feet to provide maintenance access across the weir crest. The lateral weir elevation varies from 1307.24 to 1307.00 from upstream to downstream.

5.3.3.3 Basin Outlet

The proposed basin outlet is a 2-6'x4' RCBC with flap-gates. The inlet invert is set at 1294, two feet lower than the proposed basin bottom elevation of 1296. The outlet invert is at 1293 allowing for a net 1.5-foot drop for flap gate clearance and apron slope to the proposed EMF elevation of approximately 1291.5. The outlet drains the basin within 36 hours. The model was also run to evaluate a situation in which the flap gates were blocked. In such a case, the basin fills to a peak stage of 1307.9, drains over the emergency spillway until the basin stage is 1307.0 and then ceases to drain.

5.3.3.4 Emergency Spillway

The emergency spillway is located adjacent to the EMF along the west edge of the Chandler Heights Basin just north of the basin outlet structure. It is 550 feet in length along the crest. Access across the spillway is maintained with 10:1 ramps at each end extending from the top of the embankment (el.1309) two feet down to the crest (el.1307). The spillway is protected from scour during operation by rock mattresses and a cut-off wall. It is sized to pass the peak discharge entering the basin (3,225 cfs) without overtopping the embankment (el. 1309) when the outlet structure fails to operate.

5.3.4 Channel Design

5.3.4.1 General

The channelization of Queen Creek, Sanokai Wash and a portion of the EMF is part of the overall Chandler Heights Basin system design. Channel improvements along Queen Creek and Sanokai Wash are necessary to control the lateral weir operations, provide adequate conveyance with freeboard, control channel flow velocities and control sediment transport and degradation/ aggradation in the channels. Drop structures are proposed to maintain milder, more stable channel slopes. The relocation of an existing drop structure in the EMF near Chandler Heights Road Bridge to upstream of the Chandler Heights Basin outlet has been proposed to reduce the EMF water surface elevation at the basin outlet. This will allow for gravity drainage of the detention basin.

5.3.4.2 Queen Creek

Queen Creek will be realigned and channelized from downstream of Higley Road to the confluence with Sanokai Wash. The proposed channel will be aligned just west and adjacent to the existing wash and incised to remove the levee conditions. It will have a 100-foot bottom width, 4:1 side slopes and a channel slope of 0.0003 ft/ft. The channel will be earthen except in the vicinity of six proposed weir/drop structures (Section 5.3.5).

5.3.4.3 Sanokai Wash

Sanokai Wash, between the confluence with Queen Creek and Higley Road, will consist of an earthen, incised channel located north of the proposed Ocotillo Road alignment. It will have a 110-foot bottom width, 4:1 side slopes and a channel slope of 0.0022 ft/ft. The channel contains a 5-ft drop structure and ends at Higley Road at the proposed invert for a future bridge or culvert.

5.3.4.4 Queen Creek / Sanokai Wash

From the Queen Creek and Sanokai Wash confluence to the upstream end of the lateral weir, the channel is an incised, earthen channel with a 200-ft bottom width, 4:1 side slopes and a channel slope of 0.0003 ft/ft. Along the lateral weir, the channel bottom will narrow from 200 ft to 50 ft. Within this transition, the channel side slopes will be 4:1 and the channel grade will continue at 0.0003 ft/ft. The narrowing of the channel increases the water surface elevation along the length of the weir, thus maintaining head on the weir and increasing the weir efficiency. The 50-ft wide channel downstream of the weir will outfall into a sedimentation basin prior to discharging into the EMF.

5.3.4.5 EMF

To allow for gravity drainage of the basin, the existing EMF drop structure near Chandler Heights Road will be removed and the EMF will be excavated to a new drop structure constructed upstream of the basin outlet. The EMF channel invert will be lowered approximately 5.5 feet for a distance 2700 feet upstream. The EMF bank side slopes will be extended deeper at existing 3:1 slopes thereby narrowing the bottom width through this reach from 200 feet to 167 feet. The EMF will be transitioned back to its full 200 foot bottom width near the location of the removed existing drop structure. A sloping, grouted-rock drop structure will be constructed at the new location.

5.3.5 Drop Structure Design

5.3.5.1 General

The proposed drop structures consist of an upstream constriction, approach apron, sloping drop and downstream apron with sill and cut-off walls. The upstream and downstream ends are protected with dumped riprap. The structures are comprised of grouted rock. The crest widths, drop heights and cut-off wall depths of each structure meet the needs at the given location. Drop structures were designed for a range of flow rates in order to contain potential hydraulic jumps on the drop structure apron. The design of each structure accounts for seepage, uplift forces and local scour.

5.3.5.2 Seepage and Uplift Analyses

Seepage and uplift forces were estimated using Lane's Weighted Creep as described in the "Design of Small Dams" (Bureau of Reclamation, 1973) and in accordance with the FCDMC design criteria. Based upon the analyses, the minimum thickness of each drop structure was estimated to counteract uplift forces and the depths of cut-off walls for each structure were determined to counteract seepage and piping under the structure.

5.3.5.3 Local Scour

Local scour calculations were performed and cutoff wall depths checked to insure safety against undermining of the channel structures. Scour estimates are based upon technical guidelines described in "Computing Degradation and Local Scour" (Bureau of Reclamation, 1984). Local scour is estimated based upon Type A & B Equations, the Lacey Equation, the Blench Equation and the USBR II Equation. Since these values tend to have a wide range of variability, engineering judgment was used to select the scour estimate.

5.3.5.4 Cutoff Wall Depths

Based upon the results of the seepage and local scour calculations, the required depths of cutoff walls for each drop structure were determined. A comparison of the cutoff wall depth required to counteract seepage versus local scour was made. Each cut-off wall protects against local scour. If necessary, the cutoff walls are deepened to meet seepage requirements.

5.3.5.5 Hydraulic Jump Analyses

The hydraulic jump condition at each drop structure was assessed for a range of flow-rates to obtain the minimum downstream apron length. The apron length is determined such that a hydraulic jump would be contained on the apron. HEC-RAS was used to approximate the occurrence, location and height of the jump. An estimate of the length of the jump was made using the Froude number and downstream flow depth (Chow, 1959). By modeling each drop structure over a range of flows, the 'worst case' flow condition was used as the basis for design. Typically, the 'worst case' condition was caused by a lower flow rate than the 100-year peak discharge. At high flow rates, the drop structures tend to be inundated and no jump occurs.

5.3.5.6 EMF Drop Structure

To allow for the gravity drainage of the proposed Chandler Heights Detention Basin within 36 hours, it is necessary to relocate the existing vertical drop in the EMF to upstream of the detention basin outlet. The existing concrete drop structure will be removed and a new, sloping, grouted-rock structure will be constructed upstream at the new grade control/drop structure location.

5.3.6 Sedimentation Basin Design

A new sedimentation basin is to be located just downstream of the Higley Road Bridge on Queen Creek to capture incoming sediment loads from Queen Creek and. The inlet and outlet of the sedimentation basin is protected by a grouted rock weir structure and an outlet pipe with drain filter to drain the basin. The existing sedimentation basin at the confluence of Queen Creek and the EMF will remain with some modification to the lower invert of the Queen Creek channel.

Figure 5.3.7.1 - Stage Hydrographs at EMF/Chandler Heights Outlet

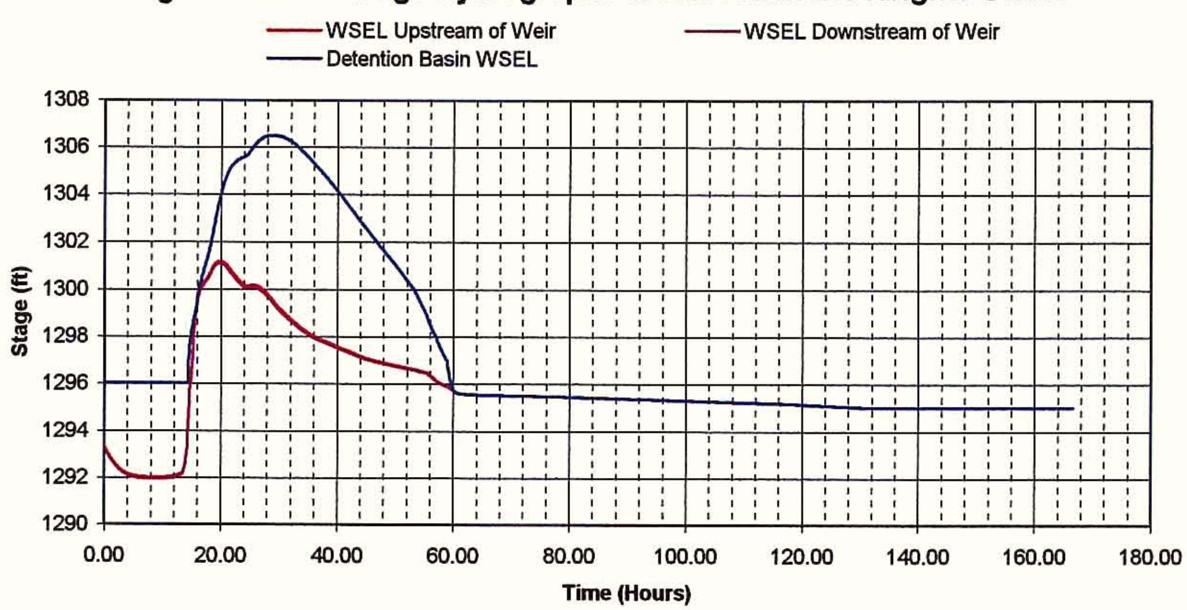
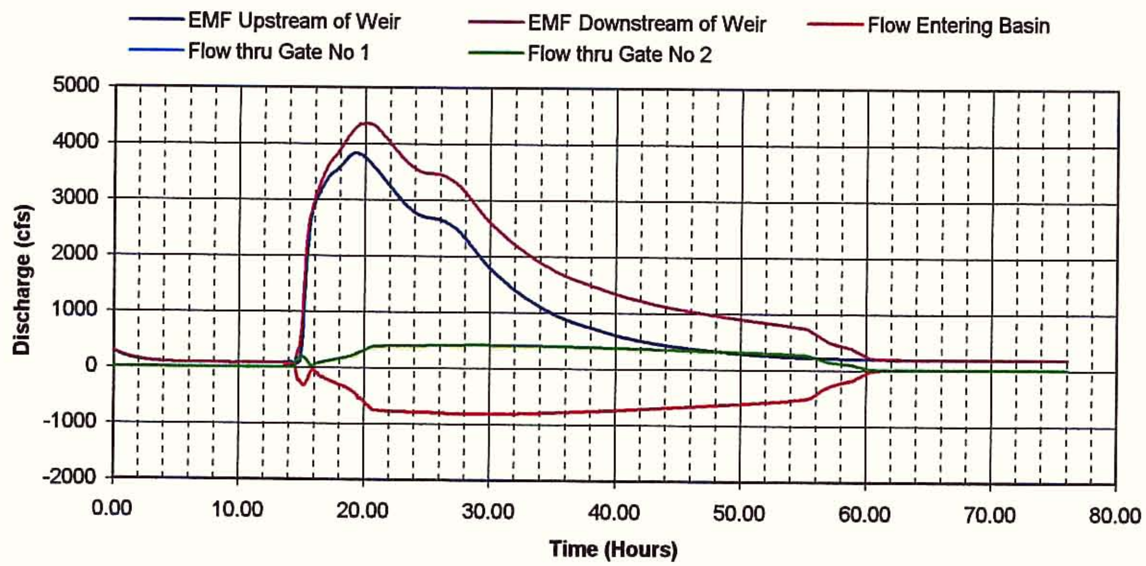


Figure 5.3.7.2 - Flow Hydrographs at EMF/Chandler Heights Outlet



5.3.7 Analysis Results

Hydrographs from the Chandler Heights Basin analysis are depicted Figures 5.3.7.1 – 5.3.7.3. Figure 5.3.7.1 shows stage hydrographs for the Chandler Heights Basin and the EMF at the basin outlet. It shows the basin stage peaking at 1306.5 for a maximum storage volume of 1320 acre-ft and providing a minimum of 2 feet of freeboard around the basin. I also shows that within 36 hours, the basin drains to an elevation of 1296.3. The remaining 0.3

Figure 5.3.7.3 - Flow Hydrographs at Weir on Queen Creek

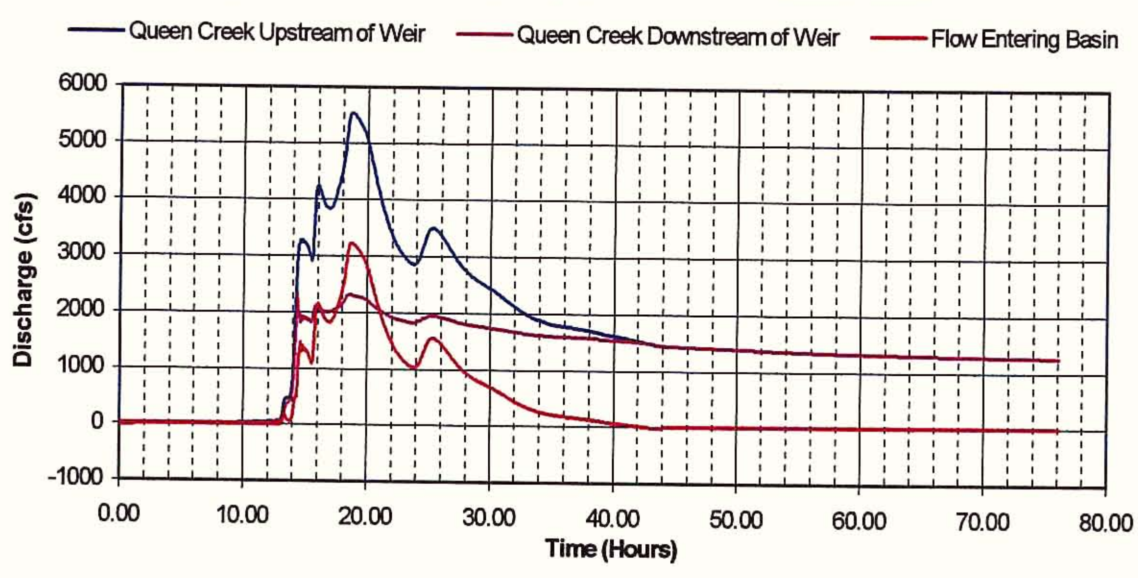
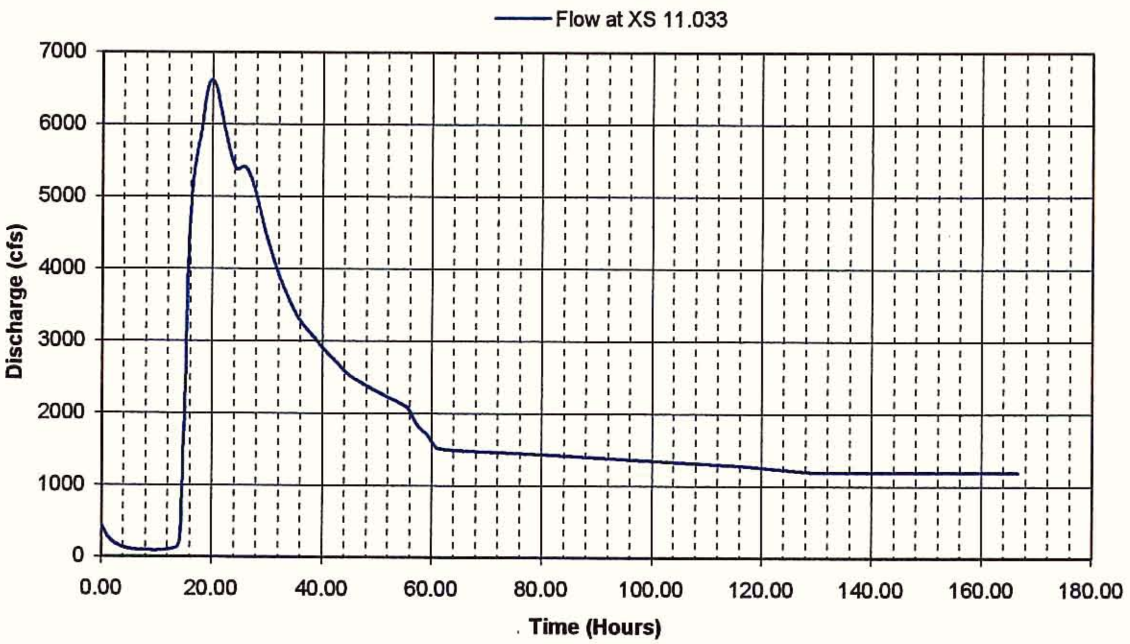


Figure 5.3.7.4 - Flow Hydrograph Downstream of Chandler Heights Basin Outlet and Queen Creek Confluence



feet of water should quickly dissipate through soil infiltration shortly, thereafter. Figure 5.3.7.2 shows the impact of the operation of the basin outlet on flow in the EMF channel. The peak flow in the EMF increases from approximately 3830 to 4370 cfs due to flow draining from the basin through the flap gates.

Figure 5.3.7.3 shows the operation of the lateral weir as it reduces the peak flow from 5540 to ~2340 cfs in Queen Creek by diverting it into the Chandler Heights Detention Basin.

Figure 5.3.7.4 shows the hydrograph in the EMF downstream of the Chandler Heights Basin Outlet and downstream of the confluence with the Queen Creek Channel. The results show the peak flow in the EMF is approximately 6610 cfs.

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