Drainage Policies and Standards for Maricopa County
Supplemental Technical Document

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FLO-2D VERIFICATION REPORT

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LIST OF TERMS AND ABBREVIATIONS

These terms and abbreviations are shown in italics in the report text.

Term	Definition
(1)	**(2)**
1D	One-dimensional model such as *HEC-RAS* or *HEC-1*.
1D Channel	A channel modeled in *FLO-2D* using one-dimensional unsteady dynamic flood routing similar to unsteady *HEC-RAS*. The channel cross sections are typically limited to the wash main channel. Overbank flood routing is modeled using the *2D* Floodplain grids.
1D Street	An urban street section with curb and gutter modeled in *FLO-2D* using one-dimensional unsteady dynamic flood routing. The street cross section is simulated with a shallow rectangular geometry.
2D	Two-dimensional model such as *FLO-2D*.
2D Floodplain	Two-dimensional overland unsteady dynamic flood routing in *FLO-2D* using the uniform square grid elements.
3D	Three-dimensional surface model of the study area used to calculate *FLO-2D* grid elevations.
ADMP	Area Drainage Master Plan
ADMS	Area Drainage Master Study
ArcGIS	An *ESRI GIS* software package.
ARS	*USDA* Agricultural Research Service
ASCII	American Standard Code for Information Interchange
BFE	Base Flood Elevation. The water surface elevation produced by a base flood or 100-year flood.
CFR	Code of Federal Regulations
CRS	FEMA Community Rating System
Dam Safety EAP	Emergency Action Plan for an FCDMC dam or flood retarding structure
DDM Hydrology	Drainage Design Manual for Maricopa County – Hydrology (FCDMC, 2013a)
DDM Hydraulics	Drainage Design Manual for Maricopa County – Hydraulics (FCDMC, 2013b)
DRI	Desert Research Institute, University of Nevada Las Vegas

Term	Definition
(1)	(2)
EPA-SWMM	US Environmental Protection Agency Storm Water Management Model
ESRI	Environmental Systems Research Institute. International supplier of Geographic Information System (*GIS*) software, and web *GIS* and geodatabase management applications.
FCDMC	Flood Control District of Maricopa County.
FDS	Floodplain Delineation Study.
FEMA	Federal Emergency Management Agency.
FHWA	United States Federal Highway Administration
FIS	Flood Insurance Study
FLO-2D	The proprietary *FLO-2D* two-dimensional flow model (computer program)
FLO-2D 2009.06	The 2009.06 version of the *FLO-2D* model (latest edition) (computer program)
FLO-2D Pro	The professional version of the *FLO-2D* model (latest edition) (computer program)
Froude number	The ratio of inertial to gravitational forces (dimensionless)
FRS	Flood Retarding Structure
GARR	Gage adjusted radar rainfall
G&A	Green and Ampt rainfall loss estimation method
G&A Computation Program	Independent computer program written by *FCDMC* to solve the G&A equation using both the *HEC-1* and *FLO-2D* source codes as an independent verification of the *FLO-2D* infiltration calculation routines. The program allows for testing infiltration from rainfall varying over time, level pool scenarios where the fixed initial volume varies with time or the head stays constant with time, and limiting infiltration depth.
GDS Pro	The *FLO-2D* Grid Development System software program
GIS	Geographical Information System
HDS-5	Hydraulic Design Series Number 5 (USDOT, 2005)
HEC-1	*USACE* Hydrologic Engineering Center *1D* hydrology model (computer program, version 4.1).
HEC-GeoRAS	*USACE* Hydrologic Engineering Center add-in to *ArcGIS* for creating *HEC-RAS* input data files (computer program, version 10.1).

Term	Definition
(1)	(2)
HEC-HMS	*USACE* Hydrologic Engineering Center Hydrologic Modeling System (computer program, version 4.1.0)
HEC-RAS	The *USACE 1D* open channel hydraulics model (computer program)
HEC-SSP	The *USACE* Statistical Software Package
HGL	Hydraulic grade line
HY-8	*FHWA* HY-8 Culvert Analysis Program (latest edition) (computer program, version 7.30)
IGA	Intergovernmental Agreement
IC	Inlet Control
Infiltration Loss	The sum of *Rainfall Loss* and *Transmission Loss*, in.
LID	*FLO-2D* limiting infiltration depth option, ft. The depth within the soil matrix corresponding to an impermeable layer.
LIDev	Low impact development.
LIDAR	Light Detection And Ranging. A technology to make high-resolution topographic maps.
Mapper	The *FLO-2D* post processor software program.
MCFCD	Mohave County Flood Control District
NEXRAD	Next-generation radar
NFIP	National Flood Insurance Program.
RMSD	Root Mean Square Deviation
NRCS	National Resources Conservation Service.
NWS	National Weather Service
OC	Outlet Control
Ponded Water, or Storage	Runoff volume that is trapped on the surface, or assumed to be trapped, and cannot be exchanged with adjacent grids, ft. Includes *TOL*.
Rainfall Loss	Portion of rainfall that is infiltrated into the soil matrix including *IA*, in
RCP	Reinforced concrete pipe

Term	Definition
(1)	(2)
SMU	*NRCS* Soil Map Unit
SSURGO	*NRCS* Soil Survey Geographical Database
StormCAD	Storm Sewer Analysis and Design Software by Bentley
SWMM	*EPA* Storm Water Management Model
Transmission Loss	Runoff volume that is infiltrated into the soil matrix after *Rainfall Loss* is accounted for, in.
USACE	United States Army Corps of Engineers
USDA	United States Department of Agriculture
USDOT	United States Department of Transportation
WSEL	Water surface elevation

LIST OF VARIABLES

Term	Definition
(1)	(2)
γ	(*PSIF* + *Head*) * *DTHETA*
φ	*PSIF*
$\Delta\theta$	*DTHETA*
ΔF	Change in infiltration over the computation time step
ΔQ_x^{i+1}	Discharge at a grid
Δt	Computation time step
Aavg	Cross sectional area, ft^2
AMANN	In the *FLO-2D* CONT.DAT file this variable increments the floodplain Manning's *n* roughness coefficient at runtime.
ARF	*FLO-2D* area reduction factor.
Aswf	Available storage area

Term	Definition
(1)	**(2)**
CN	The *NRCS* Curve Number, dimensionless.
COURANTFP	In the *FLO-2D TOL*.DAT file this variable sets the Courant number for the floodplain.
C_w	weir discharge coefficient
D_a	Average uniform flow depth, ft
depth	Parameter in the *FLO-2D* depth-variable roughness equation representing flow depth on a *2D* grid element or in a 1D channel between 0.2 ft and d_{max}.
D_f	Final depth in grid, ft
d_{max}	Parameter in the *FLO-2D* depth-variable roughness equation. *1D Channel*: Bank full flow depth, *2D Floodplain*: 3.0 feet
DTHETA	*G&A* Parameter: Volumetric soil moisture deficit, dimensionless
$d_x^{i+1}, d_x^i, d_{x+1}^i$	Flow depth on a grid
f(t)	Infiltration rate at time t
F(t)	Total infiltration at time t
H	Depth of water over the weir, ft
Head	Incremental rainfall for the time step plus flow depth on the grid element
IA	Initial abstraction or surface retention loss, in. The summation of all *Rainfall Loss* other than infiltration. It is the sum of interception, depression storage (puddles), evaporation, and evapotranspiration. For *HEC-1* and *FLO-2D*, it also includes any infiltration that occurs while *IA* is being met.
IRAINARF	In the *FLO-2D* RAIN.DAT file this variable indicates that individual grid element depth-area reduction values will be assigned.
IRAINBUILDING	In the *FLO-2D* RAIN.DAT file this variable indicates that rainfall on the *ARF* portion of the grid element will be contributed to the surface water runoff for that element.
INOUTCONT	In the *FLO-2D* HYSTRUC.DAT file set this variable to 0 to compute the discharge based on the head-water depth above the appropriate floodplain or channel bed elevation or to 1 to adjust the rating table discharge for tailwater submergence.
I_m	Infiltration volume computed by the *G&A Computation Program*, in
IWRFS	In the *FLO-2D* CONT.DAT file this variable specifies that area and width reduction factors will be assigned in the *ARF*.DAT file.

Term	Definition
(1)	(2)
K	*XKSAT*
K_w	*XKSAT*
L	Weir length, ft
L_c	Length of channel, ft
n	Manning's roughness
n_b	Parameter in the *FLO-2D* depth-variable roughness equation representing roughness at d_{max}
n_d	Parameter in the *FLO-2D* depth-variable roughness equation representing roughness at *depth*.
POROS	*FLO-2D* parameter for soil porosity.
PSIF	*G&A* Parameter: Capillary suction, in.
Q	Discharge, cfs
Qn, Qe, Qs, Qw, Qne, Qse, Qsw, Qnw	Incremental discharges for a time step across eight boundaries
R	Hydraulic radius, the cross sectional area divided by the wetted perimeter, ft
r2	Parameter in the *FLO-2D* depth-variable roughness equation representing the roughness adjustment coefficient prescribed by the user. (0. to 1.2).
RAINARF	In the *FLO-2D* RAIN.DAT file this variable is the rainfall depth area reduction to create spatially variable rainfall.
r_c	Parameter in the *FLO-2D* depth-variable roughness equation equal to $1/e^{-r2}$
R_d	Total rainfall depth, in
RTIMP	Impervious area, %.
SATI	*FLO-2D* parameter initial saturation, %.
SATF	*FLO-2D* parameter final saturation, %.
S_{hg}	Slope of the hydraulic gradeline, ft/ft
SOILD	*FLO-2D* global parameter to set limiting infiltration depth, feet.
TIME_ACCEL	In the *FLO-2D TOL*.DAT file this variable is the coefficient to increase the rate of incremental timestep change.

Term	Definition
(1)	**(2)**
TOL	*FLO-2D* Parameter: Minimum flow depth before runoff volume is exchanged with adjacent grid elements, ft. *TOL* can be used to represent the depression storage portion of *IA*.
V	Velocity, ft/sec
V_i	Total computed infiltration volume, ac-ft
W_c	Wetted perimeter of channel width, ft
WRF	*FLO-2D* width reduction factor.
XKSAT	*G&A* Parameter: Hydraulic conductivity at natural saturation, in/hr.

1 INTRODUCTION

1.1 Purpose

The purpose for this study is to provide supporting technical documentation for the Flood Control District of Maricopa County (*FCDMC*) acceptance of the *FLO-2D* model (version 2009.06 and *FLO-2D Pro*) for both hydrologic and hydraulic modeling within its jurisdiction. This document is also intended to support a request for *FEMA*'s approval of local use of *FLO-2D* for hydrologic and hydraulic modeling, engineering analysis and mapping of special flood hazard areas for the National Flood Insurance Program (NFIP). This report has been prepared by the Engineering Division of the *FCDMC* and is intended to apply within the local jurisdiction of the *FCDMC*, Maricopa County and its municipalities.

It is understood that *FEMA*'s approval of the County's local use of *FLO-2D* implies the following:

- Use of *FLO-2D* is only acceptable for drainage areas within or contributing to portions of Maricopa County.
- Professional Engineers using *FLO-2D* are ultimately responsible for the appropriate application and accuracy of the results.
- FEMA is not responsible for technical support or accuracy of the results and has not evaluated the technical soundness of *FLO-2D* for combined hydrologic and hydraulic computations.

The ultimate purpose is to show that the *FCDMC* application of the *FLO-2D* model, when following the guidance provided herein, is in compliance with the requirements of 44 *CFR* 65.6(a)(6).

Various computer models that are nationally accepted were used in the verification process. These include:

1. *HEC-RAS* version 4.1.0
2. *HEC-1* version 4.1
3. *HEC-HMS* version 3.5
4. *HY-8* version 8.7.2

1.2 Scope

The analyses and review in this report address the following versions of the *FLO-2D* model:

- *FLO-2D* Version 2009
- *FLO-2D Pro* Build 12.10.02 through Build 15.10.13

Refer to Section 6 for more detail on approved versions and any stipulations. When the model is referred to as *FLO-2D* herein, the reference includes both the 2009.06 and Pro versions. References to *FLO-2D* 2009 are specific to *FLO-2D* 2009.06 and references to *FLO-2D Pro* are specific to that version.

This investigation is intended to cover the adequacy and technical applicability of the *FLO-2D* model output for the purposes of the *FCDMC* as described herein. It is not intended to test the adequacy of the various data input tools or the available output post-processing tools. The *FLO-2D 2009* model, including previous versions, has generally been deemed acceptable for use within *FCDMC* since about 2001. The *FLO-2D Pro* model has generally been deemed acceptable for use within *FCDMC* since October 2012. This investigation documents many of the previous tests, done either as analyses or as physical tests-in-practice on actual projects during that period, plus the results of ongoing investigations and verifications.

Finally, this study lists application guidelines and identifies known limitations of the *FLO-2D* model.

1.3 Summary of Findings

FLO-2D was tested thoroughly for application as both a hydrologic and hydraulic model for use on floodplain delineation projects, area drainage master studies, dam safety hazard mapping, levee safety hazard mapping, and for general flood hazard modeling. *FLO-2D* was found to be acceptable by *FCDMC* for these purposes within its jurisdiction when properly applied.

1.4 Disclaimer

The *FCDMC* and the *FCDMC* staff do not warrant the performance and results of using the *FLO-2D* software. The application of *FLO-2D* results and the information presented in this *FLO-2D* Verification Report are the users' responsibility ONLY. The results of the testing and verification of the *FLO-2D* model are provided 'as is' without warranty of any kind, either express or implied, including, but not limited to, the implied warranties of fitness for a purpose, or the warranty of non-infringement. Although the *FLO-2D* software can be used in applications for *FCDMC* permits, using it does not guarantee that the applications will be approved. The *FCDMC* makes no warranty that:

- the *FLO-2D* software will meet your requirements.
- the quality of the *FLO-2D* software will meet your expectations.
- the application of the *FLO-2D* software will be uninterrupted, timely, secure or error-free.
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In no event shall the *FCDMC* be liable to you or any third parties for any special, punitive, incidental, indirect or consequential damages of any kind, or any damages whatsoever, including, without limitation, those resulting from loss of use, data or profits, whether or not the *FCDMC* has been advised of the possibility of such damages, and on any theory of liability, arising out of or in connection with the use of this *FLO-2D* Verification Report. The use of the *FLO-2D* Verification Report content is done at your own discretion and risk and with agreement that you will be solely responsible for any damage to your computer system or loss of data that results from such activities. No advice or information, whether oral or written, obtained by you from the *FCDMC* shall create any warranty for the use of the *FLO-2D* software or this *FLO-2D* Verification Report.

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2 TECHNICAL SUITABILITY

2.1 Purpose

The purpose of this section is to demonstrate that the *FLO-2D* model has the appropriate capabilities for performing hydrologic and hydraulic modeling for flood hazard, floodplain delineation, and dam and levee safety inundation analyses. The minimum requirements that *FLO-2D* must have to meet FCDMC needs for hydrologic modeling are listed in Section 2.3.1, and for hydraulic modeling in Section 2.4.1. The *FLO-2D* capabilities that meet or exceed these minimum requirements are described in the following sections. Refer to Section 3 for the capabilities of *FLO-2D* that make it uniquely suitable for certain aspects of hydrologic modeling, engineering analysis and mapping of Special Flood Hazard Areas. These are capabilities that other non-proprietary models may not possess. Refer to Section 4 for verification that the technical components listed and described in this section perform as stated by the software provider and meet the current standards of the *FCDMC* and Maricopa County as defined in the *Drainage Design Manual for Maricopa County, Hydrology* (FCDMC, 2013a), the *Drainage Design Manual for Maricopa County, Hydraulics* (FCDMC, 2013b), and the *Drainage Policies and Standards for Maricopa County* (FCDMC, 2012).

2.2 Overview of *FLO-2D* Program Structure

The *FLO-2D* program consists of the following:

1. Data Files. A series of ASCII data files that provide input data and parameters for a given model.
2. CONVERTER Pro. The CONVERTER Pro program converts *FLO-2D* version 2009.06 data files to the *FLO-2D* PRO version format.
3. *GDS*. A pre-processor program named Grid Developer System (*GDS Pro*) that is used to create and edit the input data files in a *GIS*-like environment. The data files can also be created using a standard text editor or using standard *GIS* programs such as *ArcGIS*. The *FLO-2D* model can be run from the *GDS* interface.
4. *FLO-2D* Executable. The *FLO-2D* program executable can be run by copying the executable program file into the folder containing the model input data files and then running it. It can also be run from the *GDS* interface. The VC2005-CON.DLL file is also required when running the *FLO-2D – SWMM* interface. It is recommended that the most current version of the VC2005-CON.DLL file also be in the model directory when running the *FLO-2D* executable from within that directory.
5. *HYDROG.* Channel output hydrographs and floodplain cross section hydrographs can be viewed with the HYDROG program.

6. MAPPER Pro. A post-processing program that creates *GIS* shape files of various result types and presents them in a *GIS*-like environment. The shape files that are created can also be opened in a standard *GIS*-based program such as *ArcGIS*.
7. MAPPER ++. MAPPER++ is a post-processor program that creates high resolution maps and plots of the *FLO-2D* model results in a *GIS*-like environment including area of inundation, time variation of hydraulic variables, maximum water surface elevations, duration of inundation, impact force, static pressure, specific energy, sediment scour or deposition and others.
8. MAXPLOT. A simple and fast program that reads *FLO-2D* output files and renders images of grid based results. MAXPLOT does not create *GIS* files.
9. PROFILES. The PROFILES program serves the dual purpose of being a pre- and post-processor program. As a pre-processor program, *1D* channel cross sections can be viewed and edited. As a post-processor program, it will display channel water surface and bed elevations for a specified *FLO-2D* simulation output interval. In order to view the predicted water surface elevations in PROFILES, it is necessary to run a *FLO-2D* channel simulation first. The PROFILES program has zoom and print options to assist in reviewing the results.
10. RAIN. The RAIN.EXE program is used to create and edit the RAIN.DAT input data file. It is most often used for assigning moving storm data.

All of the software listed, above except item 4, are pre- or post-processing programs that are not directly required to run the *FLO-2D* model. Model execution only requires the *ASCII* text input data files, the *FLO-2D* executable and possibly the VC2005-CON.DLL program library.

2.3 Hydrologic Modeling

2.3.1 General

In order to be acceptable for use in Maricopa County, the *FLO-2D* model must, as a minimum, be able to simulate the following:

1. *FCDMC* standard design storms, including temporal distributions and spatial extent.
2. Green and Ampt rainfall loss computation method as implemented by *FCDMC*.
3. Account for impervious area in rainfall loss calculations.
4. Generate runoff hydrographs at specific locations.
5. Routing of runoff hydrographs to at least the level of detail provided by HEC-1.
6. Conserve volume and provide volume accounting information.

The following sections describe the *FLO-2D* capabilities that meet the above requirements, and additional capabilities that are very beneficial to the *FCDMC* mission, which is to provide regional flood hazard identification, regulation, remediation, and education to Maricopa County residents so that they can reduce the risk of injury, death, and property damage from flooding, while still enjoying the natural and beneficial values served by floodplains.

2.3.2 Rainfall

2.3.2.1 Design Storm Rainfall

The *FCDMC* 6-hour and 24-hour design storm rainfall distributions are built into the *GDS* program. The desired storm type can be specified by the user and the correct rainfall data automatically added to the RAIN.DAT input data file. Design storm rainfall distributions can also be added manually. The total point rainfall depth is supplied along with a dimensionless temporal rainfall distribution that is applied to the entire model. This capability meets the minimum requirement.

2.3.2.2 Moving Storm Rainfall

Moving storm rainfall can be simulated through two different approaches. The first is to specify a moving design storm. The user specifies the storm direction (N, NE, E, SE, S, SW, W, or NW) and speed. The 2nd approach is to simulate real storm rainfall using Next Generation Radar (*NEXRAD*) data. *NEXRAD* grid data can be pre-processed using the *GDS* program to create a moving storm data set that essentially defines a temporal rainfall distribution for every grid in the model based on real time. This capability exceeds the minimum requirement.

2.3.2.3 Depth-Area Reduction

The *HEC-1* JD Record method of accounting for depth-area reduction of point rainfall cannot be directly implemented with a *2D* hydrologic model. *FLO-2D* does allow depth reduction to occur at the grid level. Every grid in the model can be assigned a rainfall reduction factor that the total storm point rainfall is multiplied by. The reduced rainfall on each grid is then distributed using the dimensionless rainfall distribution assigned to the model. This capability does not directly match the capability of the *HEC-1* model, but makes up for it with the spatially-varied rainfall depth reduction capability.

2.3.3 *Rainfall Loss* and Excess

2.3.3.1 Loss Estimation Methods

The *FLO-2D* model implements three (3) *Rainfall Loss* estimation methods:

1. *NRCS* Curve Number
2. Green and Ampt

3. Horton Infiltration Model (*FLO-2D* Pro Build 13.02.04 and later).

All three methods can be applied globally. All three methods can also be applied using a spatially-varied approach where the parameters can be assigned individually to each grid element. The Initial and Uniform loss method, which is a method approved for use by *FCDMC* under certain circumstances, is not implemented.

The *NRCS* Curve Number method is not normally used in Maricopa County but does have some utility on certain projects. It consists of two parameters: *CN* and *IA*. This method only provides an estimate of *Rainfall Loss* and does not simulate *Transmission Loss*. It is not applied to *1D* channels.

The Green and Ampt method (*G&A*) can be applied using the *FCDMC* approach and standard parameters. This method can be applied to floodplain grid elements and *1D Channel*s separately and provides an estimate of both *Rainfall Loss* and *Transmission Loss*. It consists of three parameters: *XKSAT*, *PSIF*, and *DTHETA*. The *DTHETA* parameter can be applied as volumetric soil moisture deficit (*FCDMC* definition) or soil moisture deficit.

The *NRCS* Curve Number method can be applied in combination with the *G&A* method. The *NRCS* Curve Number method is used to estimate rainfall loss until there is no rain on the grid. The *G&A* method is then used to compute *Transmission Loss* as long as there is flow depth on the grid element in excess of the *TOL* parameter. *TOL* is the minimum flow depth on a grid element before runoff volume is allowed to be shared with adjacent grid elements.

The Horton Infiltration Model is typically not used in Maricopa County and is not discussed further here.

These capabilities meet and exceed the minimum requirement.

2.3.3.2 Impervious Area

Impervious area (*RTIMP*) is the portion of a grid element that is impervious to infiltration. *RTIMP* is measured as a percentage and is subject to *IA*. In *FLO-2D,* a separate *RTIMP* value can be assigned to each grid element. This capability meets the *FCDMC* minimum requirement.

2.3.3.3 Impervious Sub-Strata

FLO-2D has the capability of simulating impervious substrata in the soil matrix. This is done using the limiting infiltration depth option, which is spatially varied. The existing *1D FCDMC* hydrologic method implemented with *HEC-1* does not have the ability to directly model underlying impervious sub-strata in the soil matrix. This capability is very valuable and exceeds *FCDMC* minimum requirements.

2.3.3.4 Rainfall-Runoff from Buildings

The *FLO-2D* model has the capability to simulate rainfall-runoff from buildings. Buildings are represented by Area Reduction Factors (*ARF*s) in the *FLO-2D* model. *ARF* values remove surface area from potential water storage on a grid element. Refer to Section 2.4.6 for a more complete description of *ARF*. Buildings may occupy a portion of a grid element, the entire grid element, or cover multiple grid elements.

There are two options for simulating rainfall-runoff from buildings. For both methods, the grid covering the building may be completely blocked (*ARF* =1.) or partially blocked (0<*ARF* ≤1.), or a combination of the two if multiple grids are required to define the building.

Option 1. The *FLO-2D* input control variable IRAINBUILDING is set to zero (0) (RAIN.DAT file). Rainfall-runoff from completely blocked grids is assumed to leave the model domain either in a storm drain or to storage. It is not exchanged with adjacent grids. When rainfall occurs on a partially blocked grid element, the rainfall on the entire grid element (including the portion with the assigned building *ARF* value) is accumulated on the remaining grid element surface area not covered by the building. The building portion of the grid element surface area is considered impervious and sheds rainfall but does not store water. Runoff from adjacent grids is not allowed to be stored within the *ARF* area. The accumulated rainfall depth (> *TOL* value) is then available for routing to contiguous grid elements. Again, if the grid element surface area is totally blocked (*ARF* > 0.95.), then there is no rainfall-runoff from the grid element.

Option 2. The *FLO-2D* input control variable IRAINBUILDING is set to one (1) (RAIN.DAT file). The rainfall on completely blocked grids can become runoff from the building to the adjacent downslope grids. Rainfall-runoff from the partially blocked grids is handled the same as for option one. Where there are multiple adjacent completely blocked grid elements, the rainfall is accumulated on each grid element surface and passed to contiguous grid elements within the building, based on slope. When the accumulated runoff reaches the edge of the building, it is exchanged with grids outside the building as runoff. The rainfall on an interior grid is routed to the building boundary based on the grid element elevations (ground topography) and roughness (Manning's *n*-value). Note that all building grids with *ARF* = 1 are set to an *n*-value of 0.030 at run time. This option is assumed to be representative of the shallow flow on a building roof being routed through the building's drainage system to the downspout. The user can control the drainage direction by adjusting the grid element elevations inside the building.

The capabilities of these two options meet and exceed the *FCDMC* minimum requirements.

2.3.3.5 *1D* Channel Method Infiltration Loss

The *1D* channel method discussed in Section 2.4.7.1 has the capability of simulating rainfall and transmission losses using the *G&A* method. The method works for both prismatic and natural channels. Channel infiltration can be implemented by:

1. *G&A XKSAT* averaged for a channel reach without setting a limiting infiltration depth.
2. *G&A XKSAT* parameter averaged for a channel reach including setting a limiting infiltration depth.
3. Spatially-varied *G&A XKSAT* for every channel element (cross section) with or without a limiting infiltration depth. Note that the limiting infiltration depth option for *1D Channels* was added at FLO-2FD Pro Build 13.02.04.

This approach is based on the assumption that the channel bed soils are sands and gravels and that the flow depth is much greater than the capillary suction. Therefore, *PSIF* is set to zero by default. *DTHETA* is assigned using the global parameter values and cannot be spatially-varied. To apply the *FCDMC* definition of *DTHETA*, set the *FLO-2D* parameters *SATI*, *SATF* and *POROS* as follows: *SATF* = 1.0 and *SATI* = *SATF* - *DTHETA*, where *DTHETA* is the average value for all channel reaches or elements in the model, and *POROS* = 0 or 1. When *POROS* is set equal to zero (0), *FLO-2D* does not multiply *DTHETA* by POROS. POROS can also be set equal to one (1) when volumetric *DTHETA* is used. *FLO-2D* will multiply *DTHETA* by one (1), which has the same effect as using zero (1).

To enable limiting infiltration depth, the *FLO-2D SOILD* parameter must be set greater than 0.001 feet. The limiting infiltration depth can be set independently for each channel reach, but cannot be spatially-varied by *1D* channel cross sectio4ns. When the limiting infiltration depth is set by reach, a beginning and ending infiltration rate must also be specified. These values are used to simulate infiltration into the channel banks, which can continue after the limiting infiltration depth is reached. *FLO-2D* uses an exponential decay function that decreases the rate from the initial value to the final value over a 72-hour period. This capability meets the *FCDMC* minimum requirements.

2.3.4 Inflow

The *FLO-2D* model can accept inflow hydrographs assigned to specific grid elements. The *GDS* can read *HEC-1* .HYD hydrograph files and convert them to a *FLO-2D* INFLOW.DAT input data file. This capability meets the *FCDMC* minimum requirement.

2.3.5 Hydrograph Generation

The *FLO-2D* model can generate runoff hydrographs at locations defined in the FPXSEC.DAT input data file. A selection of grids aligned perpendicular to the dominate flow direction is defined and the flow direction assigned. These are known as floodplain cross sections, and can consist of a single grid to a maximum of 1,000 grids. The program accumulates the discharge crossing each for each time step and writes it to an output file named HYCROSS.OUT. The peak discharge, time of peak, and total runoff volume is also reported. In addition, hydrographs are generated for many of the component features including hydraulic structures, *1D Streets*, *1D Channels*, levee and dam breaches, and inflow and outflow to a linked *EPA-SWMM* model. These capabilities meet the minimum requirement.

2.3.6 Hydrograph Routing

Hydrograph routing in the traditional *1D* modeling sense is not done using the *FLO-2D* model except for the *1D Channel*, street and multiple channel components. Instead, runoff volume from each grid element is routed dynamically in and/or out of adjacent grids in eight directions. The full dynamic wave form of the momentum equation is used to route flow in each direction separately. For *1D* components, flow is also routed dynamically. This approach exceeds the minimum requirement.

2.3.7 Transmission Loss

Transmission Loss is modeled using the *G&A* method for every computation time step as long as there is flow depth on the grid element in excess of *TOL* and an assigned limiting infiltration depth has not been reached. This is a capability in addition to the minimum required.

2.3.8 Volume Accounting

The *FLO-2D* model rigidly accounts for volume during each computation time step and reports the results both during run time and in the SUMMARY.OUT file after run completion. This capability meets the minimum requirement.

2.4 Hydraulic Modeling

2.4.1 General

In order to be acceptable for use in Maricopa County, the *FLO-2D* model should be capable of providing the following:

1. An accurate model of the surface that defines the watershed and the flood hazard area.
2. An accurate depiction of surface roughness at least equal to that provided by *HEC-RAS*.
3. An acceptable hydraulic computation method for unsteady flow.
4. Have suitable flow regime controls that allow forcing subcritical flow or allow supercritical flow.
5. Adequately model flow obstructions at least as well as *HEC-RAS*.

6. Have the capability of modeling *1D* open channels and hydraulic structures with functionality similar to *HEC-RAS*.
7. Have levee side weir capability.
8. Ability to model the failure of levees and dams.
9. Ability to provide the hydraulic results needed to delineate floodplain limits to *FEMA* and *FCDMC* standards.

The following sections describe the *FLO-2D* capabilities that meet the above requirements, and additional capabilities that are very beneficial to the *FCDMC* mission.

2.4.2 Surface Model

FLO-2D models are intended to be developed using the most accurate topographic mapping available to build a surface of the area to be modeled. The same mapping normally used for a *1D FDS* study may be used. *FLO-2D* is based on a uniform square grid. The average elevation of each grid is computed from the detailed mapping surface. In general, the smaller the grid size, the more accurate the model will be. However, the selection of an acceptable grid size involves a balance between accuracy of the surface depiction, model run time, depiction of significant hydraulic features such as channels, streets, and hydraulic structures, and meeting the project goals. The *FLO-2D* model provides enough diversity in components that *FEMA* accuracy requirements can be met. This is accomplished by selecting a grid size that, in combination with the use of *1D* components, provides an adequate depiction of flooding (volume distribution) on the surface to meet the project goals as well as *FEMA* minimum standards (10-meter). In many cases, selection of a small grid size (10 ft to 25ft) without *1D* channels is adequate.

2.4.3 Surface Roughness

FLO-2D has rigorous tools for defining and modeling of surface roughness summarized as follows:

1. *2D Floodplain*: Assignment of an average Manning's *n*-value to every *2D Floodplain* grid element to be used for flow depths of 3 ft and greater.
2. *1D Channel*: Assignment of an average Manning's *n*-value to every channel cross section to be used for bank full flow depth and greater.
3. *1D Channel* and *2D Floodplain*: Assignment of a shallow flow *n*-value (global, not spatially variable) for flow depths between *TOL* and 0.2 ft.
4. Depth variable *n*-values:
    A. Flow depths from 0.2 ft to 0.5 ft: 0.5 times shallow *n*
    B. Flow depths from 0.5 ft to d_{max}: varies based on the following equation:

$$n_d = n_b * r_c * e^{-(r2\frac{depth}{d_{max}})}$$

where:

d_{max} = bankfull flow depth (*1D Channels*) or 3.0 ft (*2D Floodplains*). For flow depths greater than d_{max}, n_b is used

n_b = roughness for d_{max} and greater

depth = flow depth between 0.5 ft and d_{max}

r2 = roughness adjustment coefficient (*1D Channels*: $0 < r2 \leq 1.2$ (0.4 nominal), *2D Floodplain*: 0.4)

n_d = *n*-value for *depth*

$r_c = 1/e^{-r2}$ for *1D Channel*, and 1.5 for *2D Floodplain*

This system meets and exceeds the minimum *FCDMC* requirements.

2.4.4 Hydraulic Computation Method

FLO-2D is a volume conservation model. It moves the flood volume around on a series of uniform square grid elements for overland flow or through stream segments for channel routing. Each grid element is conceptually represented as an inset octagon, allowing flow computations in or out in eight directions. Flood wave progression over the flow domain is controlled by topography and resistance to flow. The general constitutive fluid equations include the continuity equation, and the equation of motion (dynamic wave momentum equation).

Flood routing in two dimensions is accomplished through a numerical integration of the equations of motion and the conservation of fluid volume for water flood.

To summarize, the solution algorithm incorporates the following steps (FLO-2D Software, Inc., latest edition):

1. The average flow geometry, roughness and slope between two grid elements or channel cross sections are computed.
2. The flow depth d_x for computing the velocity across a grid boundary or channel cross section for the next time step (i+1) is estimated from the previous time step i using a linear estimate (the average depth between two elements):

   $$d_x^{i+1} = d_x^i + d_{x+1}^i$$
3. The first estimate of the velocity is computed using the diffusive wave equation. The only unknown variable in the diffusive wave equation is the velocity for overland, channel or street flow.
4. The predicted diffusive wave velocity for the current time step is used as a seed in the Newton-Raphson solution to solve the full dynamic wave equation for the solution velocity.

5. The discharge *Q* across the grid boundary or channel cross section is computed by multiplying the velocity by the cross sectional flow area. For overland flow, the flow width is adjusted by the width reduction factors (*WRF*s).
6. The incremental discharges for the time step across the eight boundaries (or upstream and downstream channel cross sections) are summed:

$$\Delta Q_x^{i+1} = Q_n + Q_e + Q_s + Q_w + Q_{ne} + Q_{se} + Q_{sw} + Q_{nw}$$

7. and the change in volume (net discharge x time step) is distributed over the available storage area, A_{swf}, within the grid or channel element to determine an incremental increase in the flow depth:

$$\Delta d_x^{i+1} = \Delta Q_x^{i+1} \Delta t / A_{swf}$$

where:

ΔQ_x is the net change in discharge in the eight floodplain directions for the grid element for the time step Δt between time i and i+1.

8. The numerical stability criteria are then checked for the new grid element flow depth. If any of the stability criteria are exceeded, the simulation time is reset to the previous simulation time, the time step increment is reduced, all the previous time step computations are discarded and the velocity computations begin again.
9. The simulation progresses with increasing time steps until the stability criteria are exceeded.

This hydraulic computation scheme meets the minimum *FCDMC* requirements.

2.4.5 Flow Regime Controls

FLO-2D dynamically computes flow hydraulics for the actual flow regime. The user can force subcritical flow through a global or spatially-varied limiting *Froude number* option. When a limiting *Froude number* option is exercised, the model will perform the following:

1. If the actual *Froude number* for the time step at any given location is less than the limiting value, the actual hydraulic computations are used without adjustment.
2. If the actual *Froude number* for the time step at any given location is greater than the limiting value, the *n*-value is increased a small increment and the computations restarted until the target *Froude number* is reached.

The spatially-varied option allows grid elements covering areas that can actually maintain supercritical flow, such as steep paved roads or rigid bed channels, to be modeled appropriately. Other areas that may be susceptible to supercritical flow but cannot sustain it, such as steep sand bed channels, can be modeled according to the needs of the study or design project. This also allows the user to model floodplain areas as subcritical or critical flow in conformance with *FEMA* requirements. Although this is a unique approach, *FCDMC* accepts this method as reasonable.

2.4.6 Flow Obstructions

Flow obstructions in the *FLO-2D* model can be simulated by three methods:

1. Area Reduction Factors (*ARF*),
2. Width Reduction Factors (*WRF*), and
3. Levees.

An *ARF* factor has the effect of reducing the available storage on a grid element. The storage can be completely removed, making the grid element unavailable to accept flow from adjacent elements, or reduced by a percentage. An *ARF* factor can be applied to any grid element in the model unless it conflicts with another component.

A *WRF* factor has the effect of reducing the width of a specific octagonal side of a grid element and thereby reducing the cross sectional area available to convey flow in or out of a grid element. Any one of the eight sides of a grid element can be completely or partially blocked using this factor unless it conflicts with another component.

Building obstructions modeled using the *ARF* factor can be failed using a building collapse routine. By assigning negative *ARF*-values for either totally or partially blocked elements in *ARF*.DAT, the removal of buildings during the flood event can be simulated when velocity and depth criteria for the collapse of the buildings is exceeded.

A levee functions similar to a *WRF* factor of 1.0 except that additional capabilities are available. Refer to Section 2.4.9. If a specific side of a grid element is assigned as a levee, it can be overtopped or failed using both a horizontal and/or vertical failure rate. Several other functions are also available.

The *FLO-2D* flow obstructions capabilities meet the minimum *FCDMC* requirements.

2.4.7 One Dimensional Components

2.4.7.1 Open Channels

FLO-2D has open channel modeling capabilities similar to *HEC-RAS*. The *GDS* can directly import an existing *HEC-RAS* model that is properly geo-referenced and create the necessary *FLO-2D* input files. Cross sections can be viewed, modified, plotted, and interpolated. The *FLO-2D 1D Channel* implementation is intended for modeling the main channel and not the overbank conveyance areas. Those areas are normally modeled using the *2D Floodplain* grids, but the routine will model the entire floodplain if desired. Only one average *n*-value is allowed per cross section, so modeling the entire floodplain with the *1D Channel* routine may not be appropriate for a given study. Since most *HEC-RAS* floodplain models use a single average *n*-value for the main channel, the *FLO-2D 1D Channel* capability meets the minimum *FCDMC* requirements.

2.4.7.2 Streets

The hydraulic effects of streets can be modeled in *FLO-2D* as simple *1D* shallow rectangular channels. The component is normally used when the grid element is wider than the street width. The *FLO-2D* hydraulic street component capability meets the minimum *FCDMC* requirements.

2.4.7.3 Multiple Channels

This component is for the situation where one or more small channels (natural or constructed) exist within the grid element width and have significant conveyance or effect on travel times that should be accounted for. The user can simulate multiple small rectangular channels that can be allowed to widen due to bank erosion. The multiple channel component has the effect of concentrating the discharge and may improve the timing of runoff routing. This component can be used to help offset flood routing velocity issues that arise from use of a grid size that is too large to capture the hydraulic effects of small inset channel systems.

If the flow rate exceeds the specified channel flow depth, the multiple channels can be expanded by a specified incremental width. This channel widening process assumes these are alluvial channels that will widen to accept more flow as the flow reaches bank full discharge. There is no channel overbank discharge to the overland surface area within the grid element. The channel will continue to widen until the channel width exceeds the width of the grid element, then the flow routing between grid elements will revert to sheet flow. This enables the grid element to be overwhelmed by flood flows. During the falling limb of the hydrograph when the flow depth is less than 1 ft (0.3 m), the channel width will decrease to confine the discharge until the original width is again attained. The user can also assign the range of slope where multiple channel widening is computed. Additionally, the user can also specify that the predefined channel widths are fixed (no widening occurs). For this case, there is no channel overbank discharge to the available overland surface area within the grid element. Flow will be kept within the defined multiple channels, vertically increasing the available depth as required.

The hydraulics are based on the wetted perimeter only including the multiple channel bottom width. For small shallow channels (1-2 feet of depth), this approach produces acceptable results. The multiple channel approach may not be an appropriate choice when modeling channels with a fixed width and large flow depths. Instead, the prismatic *1D* channel option may be a more appropriate choice.

The *FLO-2D* multiple channel component capability meets the minimum *FCDMC* requirements.

2.4.8 Hydraulic Structures

2.4.8.1 Culverts and Bridges

FLO-2D can model culverts, bridges and spillways using hydraulic rating tables or curves, and culverts using the equations from the *USDOT HDS-5* publication *Hydraulic Design of Highway Culverts* (USDOT, 2005). Hydraulic rating tables or curves must be prepared separately by the user using other software such as *HEC-RAS* or *HY-8*. They typically should be based either on inlet or outlet control datasets with the head based on the inlet head. The rating table approach has the following additional controls using the INOUTCONT(I,J) variable:

INOUTCONT(I,J) = 0; to compute the discharge based on only the headwater depth above the appropriate floodplain or channel bed elevation (or reference elevation if assigned). Suggested revisions, generated at runtime, are listed in the REVISED_RATING_TABLE.OUT file. No tailwater effects or potential reverse flow are considered, but the rating table values can include predefined outlet control effects.

INOUTCONT(I,J) = 1; reduced discharge for tailwater submergence, but does not allow reverse flow. Suggested rating table revisions, generated at runtime, are posted to the REVISED_RATING_TABLE.OUT file.

INOUTCONT(I,J) = 2; reduced discharge for tailwater submergence. Reverse flow is possible. Suggested rating table revisions, generated at runtime, are posted to the REVISED_RATING_TABLE.OUT file.

Culverts modeled using the *HDS-5* general culvert equations are checked for both the inlet or outlet control conditions and the highest head lowest discharge result is used. There are additional capabilities under this routine that are also helpful for addressing long culverts or providing an approximate solution for storm drain systems. These capabilities meet the minimum *FCDMC* requirements.

2.4.8.2 Storm Drains

The *FLO-2D Pro* model is integrated with the *EPA-SWMM* Version 5 storm drain model (Rossman, 2010). The integration allows *FLO-2D* to control computation of the amounts of storm water the storm drain inlets can convey into the pipe system. The *EPA-SWMM* model then performs pipe hydraulics. If the pipe system cannot convey the inlet flow, then excess flow is either left on, or returned to, the surface. Other critical interface points including junction manholes and outlets/outfalls are modeled in a similar manner. Manhole covers can be removed due to a predefined setting pressure head setting and flow can then move in and out of the system through the manhole opening. Flow can exit or enter the storm drain system through outlets/outfalls based on the water surface elevation at the outlet. The *GDS* provides tools for creating the needed interface data and visualizing the storm drain system. Standard *EPA-SWMM* tools may be used to create the system. This interface meets the minimum *FCDMC* requirements.

2.4.9 Levees and Dams

2.4.9.1 Simulation Method

The *FLO-2D* levee component confines flow on the floodplain surface by blocking one or more of the eight flow directions. Levees are designated at the grid element boundaries. If a levee runs through the center of a grid element, the model levee position is represented by one or more of the eight grid element boundaries. Levees often follow the boundaries along a series of consecutive elements. A levee crest elevation can be assigned for each of the eight flow directions in a given grid element. The model will predict levee overtopping. When the flow depth exceeds the levee height, the discharge over the levee is computed using the broad crested weir flow equation. Weir flow occurs until the tailwater depth is 85% of the headwater depth. Above 85%, the water is exchanged across the levee using the difference in water surface elevation. Levee overtopping will not cause levee failure unless the failure or breach option is invoked. This component meets the minimum *FCDMC* requirements for simulation of levee blockages and overtopping (side weir).

2.4.9.2 Failure Mechanisms

FLO-2D can simulate levee and dam breach failures. There are two failure modes; one is a simple uniform rate of breach expansion and the other predicts breach erosion of an earthen embankment. In addition, *FLO-2D* can locate the levee breach based on a user specified water surface elevation and duration. For both cases, the breach time step is controlled by the flood routing model. *FLO-2D* computes the discharge through the breach, the change in upstream storage, the tailwater and backwater effects, and the downstream flood routing. Each failure option generates a series of output files to assist the user in analyzing the response to the dam or levee breach. The model reports the time of breach or overtopping, the breach hydrograph, peak discharge through the breach, and breach parameters as a function of time. Additional output files that define the breach hazard include the time-to-flow-depth output files that report the time to the maximum flow depth, the time to one foot flow depth and time to two foot flow depth which are useful for delineating evacuation routes.

The BREACH model (Fread, 1998) is the basis for the breach component. The BREACH model code was obtained from the National Weather Service (*NWS*) and extensively revised by *FLO-2D* Software, Inc. There were some code errors in the original BREACH model that were corrected as a part of this process. In *FLO-2D*, a dam or levee breach can fail as follows:

1. Overtopping and development of a breach channel,
2. Piping failure,
3. Piping failure and roof collapse and development into a breach channel,
4. Breach channel enlargement through side slope slumping, and
5. Breach enlargement by wedge collapse.

This component meets the minimum *FCDMC* requirements.

2.4.10 Mud and Debris Flow

FLO-2D has the capability of routing mud and debris flow on the *2D Floodplain*. *FLO-2D* routes mudflows as a fluid continuum by predicting viscous fluid motion as a function of sediment concentration. A quadratic rheological model for predicting viscous and yield stresses as a function of sediment concentration is employed and sediment continuity is observed. As sediment concentration changes for a given grid element, dilution effects, mudflow cessation and the remobilization of deposits are simulated. Mudflows are dominated by viscous and dispersive stresses and constitute a very different phenomenon than those processes of suspended sediment load and bed load in conventional sediment transport. The sediment transport and mudflow components cannot be used together in a *FLO-2D* simulation.

When routing the mud flood or mudflow over an alluvial fan or floodplain, the *FLO-2D* model preserves continuity for both the water and sediment. For every grid element and time step, the change in the water and sediment volumes and the corresponding change in sediment concentration are computed. At the end of the simulation, the model reports on the amount of water and sediment removed from the study area (outflow) and the amount and location of the water and sediment remaining on the fan or in the channel (storage). The areal extent of mudflow inundation and the maximum flow depths and velocities are a function of the available sediment volume and concentration which can be varied in the *FLO-2D* simulations.

This verification report does not include numerical verification of the mud and debris flow component. That is planned for a future report update, but is not needed for *FEMA* approval.

2.4.11 Sediment Transport

To address mobile bed issues, *FLO-2D* has a sediment transport component that can compute sediment scour or deposition. Within a grid element, sediment transport capacity is computed for either *2D*

Floodplain, *1D Channel*, or street based flow hydraulics. The sediment transport capacity is then compared with the sediment supply and the resulting sediment excess or deficit is uniformly distributed over the grid element potential flow surface using the bed porosity based on the dry weight of sediment. For surveyed *1D Channel* cross sections, a non-uniform sediment distribution relationship is used. There are eleven sediment transport capacity equations that can be applied in the *FLO-2D* model. Sediment routing by size fraction and armoring are also options. Sediment continuity is tracked on a grid element basis.

During a *FLO-2D* flood simulation, the sediment transport capacity is based on the predicted flow hydraulics between *2D Floodplain* or *1D Channel* elements, but the sediment transport computation is uncoupled from the flow hydraulics. Initially the flow hydraulics are computed for all the grid and channel elements for the given time step and then the sediment transport is computed based on the flow hydraulics for that time step. This assumes that the change in channel geometry resulting from deposition or scour will not have a significant effect on the average flow hydraulics for that time step. If the scour or deposition is less than 0.10 ft (0.3 m), the sediment storage volume is not distributed on the bed, but is accumulated. Generally it takes several time steps on the order of 1 to10 seconds to accumulate enough sediment so that the resulting deposition or scour will exceed 0.10 ft (0.03 m). This justifies the uncoupled sediment transport approach used in *FLO-2D*.

FLO-2D calculates the sediment transport capacity using each equation for each grid element and time step. The user selects only one equation for use in the flood simulation, but can designate one floodplain or channel element to view the sediment transport capacity results for all the equations based on the output interval. The computed sediment transport capacity for each of the eleven equations can then be compared by output interval in the SEDTRAN.OUT file. From this file, the range of sediment transport capacity, and those equations that appear to be overestimating or underestimated the sediment load, can be determined.

This component can be useful for *ADMS* studies and similar projects. These capabilities meet the minimum *FCDMC* requirements, but this verification report does not include numerical verification of the sediment transport component. That is planned for a future report update, but this functionality is not needed for FEMA approval.

2.4.12 Floodplain Limits Delineation

The *GDS* can automatically delineate floodplain limits for confined flow, similar to the *HEC-GeoRAS* capability. This can also be done using standard *GIS* tools by creating a surface from the maximum water surface elevation grid data and intersecting it with the ground surface. These capabilities meet the *FCDMC* requirements.

2.4.13 Floodway Delineation

There is currently not an automated method for delineating a regulatory floodway with the *FLO-2D* model that meets *FEMA* requirements. The *FEMA* definition from 44 CFR 59.1 reads: *"'Regulatory floodway' means the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height."* *FLO-2D* does have a floodway computation option that may have merit for a riverine system and appears to meet the *FEMA* definition, but it does not meet the strict *FEMA* interpretation of being based on equal conveyance encroachment. It is possible to perform a brute force approach requiring multiple model runs and manual definition and re-definition of encroachment limits. The *FLO-2D* floodway tool could be used for definition of an administrative floodway (FCDMC, 2014). Application of appropriate depth-velocity criteria using the *FLO-2D* results could also be used for definition of administrative floodways. The *FLO-2D* model results used to delineate AE flood zones in combination with or without administrative floodways does have utilization for *FCDMC* floodplain delineation purposes. The *FLO-2D* results can also be used in conjunction with the *HEC-RAS* model when a floodway delineation is needed.

2.4.14 Summary and Conclusions

The *FLO-2D* model has the minimum capabilities needed by *FCDMC* for floodplain delineation studies. Refer to Section 4 for verification of the validity of *FLO-2D* model results for the critical components.

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3 UNIQUE CAPABILITIES

3.1 Purpose

The *FLO-2D 2009.06* model is a non-proprietary model, but the *FLO-2D Pro* model is proprietary. *FLO-2D* 2009.06 is currently approved by *FEMA* for use as a hydraulic model but not for hydrology (http://www.fema.gov/hydraulic-numerical-models-meeting-minimum-requirement-national-flood-insurance-program). *FLO-2D Pro* is currently not on the *FEMA* approved list of numerical models for either hydrology or hydraulics. The purpose of this section is to document that the *FLO-2D* 2009.06 and *FLO-2D Pro* models provide capabilities beyond other non-proprietary hydrologic models, and that *FLO-2D Pro* provides capabilities beyond other non-proprietary hydraulic models, on the existing *FEMA* accepted models list. The intent is to obtain approval by *FEMA* for use of the *FLO-2D* model as a hydrologic and hydraulic modeling tool within the jurisdiction of the *FCDMC*.

3.2 Combined Hydrologic and Hydraulic Modeling

Both *FLO-2D 2009.06* and the *FLO-2D Pro* model perform integrated *2D* hydrologic and hydraulic computations. None of the non-proprietary models on the *FEMA*-approved numerical models list have this capability. An added bonus is that both models implement the *G&A* method and compute *Transmission Loss*. This capability is extremely important for accurately determining the volume of flood water on the surface in sheet flow (shallow unconfined flow), distributary flow areas, and urban areas where the watershed boundaries are not well defined.

In addition, it is virtually impossible to accurately estimate runoff hydrographs within such areas using the *1D* models on the list of *FEMA*-approved hydrology models. The *1D* sub-basin concept assumes that a single point of concentrations exists, which is usually not the case with sheet flow or distributary flow. Both *FLO-2D* models do an excellent job for these situations, as well as for tributary dendritic watersheds.

3.3 Integrated Flow Obstruction and Rainfall-Runoff Modeling

Proper modeling of flow obstructions in an urban area where the natural drainage system no longer exists and sheet flow and/or distributed flow dominates is very important. The important flow obstructions are buildings and masonry walls, which in conjunction with streets often control the drainage patterns and distribution of runoff volume on the surface. Attempting to appropriately model these watersheds with a *1D* hydrology model is very difficult because lumped parameter sub-basins covering relatively large areas (usually at least 0.25 sm) do not capture these flow patterns.

In addition to modeling buildings as obstructions to flow, both *FLO-2D* models simulate rainfall-runoff from buildings concurrently. This is a very important capability for *FCDMC* requirements. Also, building obstructions modeled using the *ARF* factor can be failed using a building collapse routine, if appropriate.

3.4 Spatially-varied Limiting Infiltration Depth

This capability was added to the *FLO-2D Pro* model at the *FCDMC's* request. This option allows the modeler to assign a depth below the assigned ground elevation of every grid element. That depth represents the height of the soil column that can infiltrate water. This allows for simulation of bedrock, caliche, groundwater or other impermeable layers. It is a very important option for *2D* models where both *Rainfall Loss* and *Transmission Loss* are accounted for. It is necessary in order to help prevent over estimation of *Infiltration Loss* and can also be used as a calibration tool.

3.5 Spatially-varied *TOL*

The *TOL* parameter, which is the minimum depth that must be met or exceeded before volume can be exchanged with adjacent grid elements, can be spatially-varied or applied using a single global parameter. This is very important for modeling Low Impact Development (*LIDev*) infrastructure because storage can be added on an individual grid element basis, in combination with *IA*, to simulate the *LIDev* function. This is a very valuable capability for *FCDMC* urban planning and assessment of *LIDev* flood mitigation measures.

3.6 Integrated Storm Drain Modeling

The *EPA SWMM* Version 5 *FEMA*-approved storm drain model is integrated with the *FLO-2D Pro* model. The interface allows *FLO-2D Pro* to send water from the surface into the storm drain system or receive water onto the surface from the storm drain system. This can happen at:

- street inlets and catch basins,
- pipe openings similar to culvert inlets,
- storm drain manhole covers that are open or have the covers removed by hydrostatic pressure from within the storm drain system,
- storm water storage basins connected to the storm drain system, and
- storm drain system outfalls that empty onto the *2D Floodplai*n or *1D Channel*.

FLO-2D controls the time step. Since the *EPA-SWMM* time step is typically longer than *FLO-2D*, *FLO-2D* uses the average depth on the grid element for the time steps computed while waiting on *EPA-SWMM*. *FLO-2D Pro* computes the flow rate into or out of the opening based on HEC-22 (USDOT, 2013) or

HDS-5 (USDOT, 2005). The user may also provide a rating table prepared manually using an appropriate reference or computer program, such as *StormCAD* (Bentley, 2016), *HY-8* (USDOT, 2009), *HEC-RAS* (USACE, 2010a), or hand calculations.

This capability is very important for *FCDMC* projects. It is being used in the *FCDMC* planning studies to determine the impact of existing storm drain systems on surface flooding conditions and for flood mitigation by modeling proposed storm drain systems to determine how they will function under design and 100-year storm conditions. The use of this tool allows optimizing the design of major storm drain systems so that they are not over or under-designed, thus saving tax payer dollars.

3.7 Spatially-varied Limiting *Froude Number*

This option allows the user to assign a different limiting *Froude number* to each grid element. Diverse surface characteristics such as lined channels and streets can be allowed to have supercritical flow if appropriate, while movable bed channels can be forced subcritical where appropriate. This is a very powerful tool that helps to appropriately model overall system response to rainfall events by providing more accurate velocity estimates at discrete locations.

3.8 Integrated *NEXRAD* and Moving Storm Support

3.8.1 Integrated *NEXRAD*

The *FLO-2D 2009.06* and *FLO-2D Pro* models have the capability of modeling actual storm events using grid-based temporal rainfall data from the *NWS NEXRAD* data sets. The *GDS* has a tool to read these data files and create a *FLO-2D* input data file that assigns a rainfall depth to every grid element for each uniform time step available from the *NEXRAD* data. This has the effect of applying a separate rainfall distribution to every grid element, thereby simulating a moving storm. *FCDMC* has been contracting with weather data providers that are professional hydro meteorologists to post-process the *NEXRAD* data and filter out errors and anomalies and then adjust the radar rainfall estimates to ground-based rainfall measurements from the *FCDMC* and others rain gages. This information is then applied within the *FLO-2D* model to simulate a real storm and check the results against known flood depths and available stream gage data. The model can be verified and potentially calibrated when sufficient data is available. The results can then be used to provide a more accurate planning or design storm model and to build confidence in users that the model can simulate actual conditions within the study area. This is a very important unique capability for *FCDMC* projects.

3.8.2 Moving Storm Support

The *FLO-2D* model can simulate a moving design storm in addition to a fixed design storm. This capability allows the user to set a storm direction and velocity. A design storm can then be set to mimic actual typical storm movement patterns for a watershed in order to mimic anticipated normal conditions as much as possible. This tool would typically be applied to large watersheds when setting a design storm over every square foot of watershed is not a realistic approach. It allows the user to experiment and determine what design storm size, path and velocity produce the worst flooding conditions in the area of interest. This approach also has application in flood warning scenarios for applying an approaching storm to a watershed that the storm will possibly impact.

3.9 Stage-Storage Curves by Grid Element

FLO-2D Pro has the capability of creating a stage versus storage curve for every grid element using the topographic surface that the average grid element ground elevation was computed from. This allows the model to account for actual storage within each grid element. Storage available within each grid below the assigned elevation is filled first. Although usually a fairly minor adjustment, this approach provides a more accurate volume accounting that can be significant when examining overall attenuation due to storage routing.

3.10 Integrated Levee and Dam Breach Modeling

The *FLO-2D 2009.06* and *FLO-2D Pro* models can simulate the effects of levee failure and erosion breach of earthen dams as described in Section 2.4.9. This allows for a fully coupled model where the watershed hydrology is integrated with the hydrologic storage and hydraulic obstruction effects of the levee and/or dam, the mechanisms leading up to failure of the structure or structures, the duration and extent of that failure or failures, and the hydraulics of the breach flood waves as they progress downstream. This capability is unique because of the integrated hydrologic and hydraulic model effects on and resulting from multiple levee and dam breach locations that can occur within the same model domain. Thus, a failure can be triggered by the model (not necessarily at a predefined location) based on temporal and hydraulic criteria. The results of that failure may trigger a series of other failures downstream, or keep failures from occurring at other locations. This makes for a very powerful tool for identifying possible flood hazards in a community.

Levee and dam failures can be triggered at multiple locations, simultaneously if necessary, by the following mechanisms:

1. Levees
    - A. Overtopping,
    - B. Water surface elevation reaching a prescribed distance below the levee crest elevation for an assigned duration at a specific location, or determined by *FLO-2D* using global parameters,
    - C. The above in combination with *USACE* fragility curve assignments (Schultz, Gouldby, Simm, & Wibowo, 2010).
2. Dam Erosion Breach
    - A. Predefined location and elevation,
    - B. Location not predetermined using global settings for water surface elevation and duration of inundation,
    - C. Overtopping and development of a breach channel,
    - D. Piping failure,
    - E. Piping failure and roof collapse and development into a breach channel,
    - F. Breach channel enlargement through side slope slumping, and
    - G. Breach enlargement by wedge collapse.

These capabilities have proven very useful for *FCDMC* projects including *FDS*, *ADMS*, and Dam Safety Emergency Action Plan (*Dam Safety EAP*) updates.

3.11 Summary and Conclusions

Both the *FLO-2D 2009.06* and *FLO-2D Pro* models have unique capabilities described above that have become very important for *FCDMC* purposes. These capabilities are helping *FCDMC* staff provide more accurate estimates of flood hazards for all portions of a study area, not just the confined flow locations along well-defined washes and rivers. This in turn helps the citizens of Maricopa County by:

1. not overstating or understating the flood hazard, which saves money on flood insurance,
2. helps reduce flood insurance costs through improved *CRS* rating, and
3. helps ensure that tax dollars spent on flood hazard mitigation are minimized and focused where they provide the most benefit.

An added benefit is that the municipalities and their engineering consultants can use the information from the *FLO-2D* models to locate where drainage improvements are needed to resolve local drainage issues and to test those solutions to be sure they will work and not cause a new drainage issue somewhere else. The consulting engineering community can use this information to help plan and design developments and other infrastructure improvements. The *FCDMC* has embraced this technology because of these benefits to the citizens we serve.

4 VERIFICATION

4.1 Purpose

The purpose of this section is to report on testing of numerical accuracy of the various *FLO-2D* capabilities described in Section 2 and Section 3. The following are several types of tests that were performed:

1. comparison of results with those from another model (i.e. typically HEC-1 or HEC-RAS),
2. comparison with independent manual calculations,
3. simulation of an actual storm event and comparison of results with gage measured data, and
4. review of the program source code.

Some components were tested with all testing types if the needed information was available and the component was of special importance to *FCDMC*. For some components, testing with only one or two types was practical or necessary.

Small *FLO-2D* model(s) were prepared to isolate the component being tested for Types 1 and 2. This was done to minimize the effects of interaction between components and to simplify testing. There is a need to test the model when various components are used simultaneously and interacting with each other. Type 3 was relied on to meet this verification need. In addition, where the capability being tested is available in both *FLO-2D 2009.06* and *FLO-2D Pro,* verification tests were done for both models and the results compared. The comparisons are not documented in this report unless an unacceptable difference was noted.

Numerical testing was done for the following:

1. rainfall loss using *G&A*,
2. hydraulic calculations between grid elements,
3. *1D Channel* hydraulics,
4. flow obstructions,
5. hydraulic structures,
6. levees,
7. multiple channels,
8. *EPA-SWMM* component, and
9. real storm modeling.

The verification models are the basis of a series of test models that will be used to verify that new builds of *FLO-2D Pro* function as expected without introducing unintended errors in existing capabilities.

Finally, the suitability of *FLO-2D* for delineation of floodplain limits is examined, and then the results of this section are summarized and conclusions stated.

4.2 Testing of Numerical Accuracy

4.2.1 Rainfall Losses Using Green & Ampt

4.2.1.1 General

The *G&A* method comes from a paper published in 1911 titled *Studies on Soil Physics, Part I the Flow of Air and Water through Soils* (Green & Ampt, 1911). The method set forth in that paper had limited application for many years because the physical soil data for assigning parameters was not available for large geographical areas. The *USDA* Agricultural Resource Service (*ARS*) and others have performed active research on this method since the 1960's (Bouwer, 1966) (Mein & Larson, 1973) (Mein & Farrell, 1974) (Ahuja, 1974) (Rawls, Brakensiek, & Miller, 1983) (Saxton & Rawls, 2006). The method has become viable for watershed hydrology applications with the advent of the *NRCS* detailed soil surveys of the United States. In 1990, the *FCDMC* adopted the *G&A* method in the Drainage Design Manual for Maricopa County (*DDM Hydrology*) (FCDMC, 2013a). The *FCDMC* approach to assigning parameters for use by the *G&A* method is based on *Green-Ampt Infiltration Parameters from Soils Data* (Rawls, Brakensiek, & Miller, 1983). This approach has been implemented since 1990 using the *USACE HEC-1* computer program (USACE, 1998), which uses a simplified solution of the *G&A* method derived from *Modeling Infiltration During a Steady Rain* (Mein & Larson, 1973). The *HEC-HMS* model (USACE, 2000), which is a successor to *HEC-1*, uses a different approach based on *Flood-Runoff Analysis EM 1110-2-1417* (USACE, 1994).

The *FLO-2D* model uses a solution of the *G&A* method based on *Water and Sediment Routing from Complex Watersheds and Example Application to Surface Mining* (Fullerton, 1983), which is a different approach than taken by either *HEC-1* or *HEC-HMS*. Refer to *Applied Hydrology* (Chow, Maidment, & Mays, 1988) for a detailed description of the theory the Green and Ampt method is based on. *HEC-1 and HEC-HMS* were used in tests for comparison with *FLO-2D* results. Refer to the *HEC-1 Flood Hydrograph Package User's Manual* ((USACE, 1998), the *Hydrologic Modeling System HEC-HMS Technical Reference Manual* (USACE, 2000), and the *USACE Flood-runoff Analysis*, EM 110-2-1417 (USACE, 1994) for a description of the technical basis for implementation of the *G&A* method in the *HEC-1* and *HEC-HMS* programs. The approaches used in the test programs for solution of the *G&A* equation are described in Sections 4.2.1.2 through 4.2.1.4. This information is provided so the reader can better understand the differences in approach between *FLO-2D* and the check models. This will help with understanding why there are differences in modeling results.

The results of the various tests of the *FLO-2D G&A* implementation are presented in Sections 4.2.1.5 through 4.2.1.10. *FLO-2D* results are compared with *HEC-1* and *HEC-HMS* in Section 4.2.1.5. Testing of infiltration for the *1D* channel option is covered in Section 4.2.1.7. Testing of the Limiting Infiltration Depth option is described in Section 4.2.1.9. Testing of rainfall-runoff from impervious building obstructions is contained in Section 4.2.1.10.

4.2.1.2 HEC-1 Approach

The *G&A* infiltration function (see Mein and Larson, 1973) is combined with an initial abstraction to compute rainfall losses. The form of the *G&A* equation used is:

$$F(t) = \frac{\varphi \Delta\theta}{[f(t)/K] - 1}$$

where:

$f(t)$ = infiltration rate at time t

$K = XKSAT$

φ = PSIF

$\Delta\theta = DTHETA$

$F(t)$ = total infiltration at time t

The process followed by *HEC-1* is as follows:

1. *IA* Not Satisfied. The initial abstraction has to be satisfied prior to commencing rainfall infiltration calculations. Any infiltration that may actually occur while *IA* is being met is assumed to be included in *IA*. For each time step, incremental rainfall is summed until the total equals or exceeds *IA*. The difference between total rainfall and *IA* for that time step is noted.
2. Pre-Ponding Infiltration. *HEC-1* uses a modified form of the *G&A* equation to estimate the time to ponding for each computation time step assuming a uniform rainfall rate during the time step. Ponding occurs after *IA* is satisfied and the rainfall rate is greater than or equal to the infiltration rate. If ponding will not occur, then all rainfall during the time step is infiltrated. If ponding is expected to occur during the time step all rainfall is infiltrated up to that time and ponding is verified by calculation.
3. Ponding has Occurred. After the time of ponding, the *G&A* equation is applied to compute the infiltration during each time step and excess rainfall becomes runoff. The *G&A* equation is applied until the potential infiltration rate is less than *XKSAT*. Then *XKSAT* is used to compute infiltration.

HEC-1 uses an implicit scheme to solve the *G&A* equation because it is a nonlinear equation in terms of infiltration. *HEC-1* does not include a head (flow depth) term in the *G&A* solution. Since this is a *1D* hydrology model that is only estimating *Rainfall Loss*, the head term is assumed to be insignificant when compared to the suction head (*PSIF*) at the wetting front.

4. Ponding Not Maintained. If the rainfall rate becomes less than the ponded infiltration rate, *HEC-1* reverts back to Step 2 above. *HEC-1* does not account for soil moisture recovery.

4.2.1.3 HEC-HMS Approach

The *HEC-HMS* implementation of the *G&A* method is almost identical to that used by *HEC-1*. The main difference is in how the *IA* is modeled. In *HEC-HMS*, *IA* is divided into two components: 1) Canopy-interception storage and 2) Surface-interception storage.

<u>Canopy-interception storage</u>

Canopy interception represents precipitation that is captured on trees, shrubs, and grasses, and does not reach the soil surface. Precipitation is the only inflow into this layer. When precipitation occurs, it first fills canopy storage. Only after this storage is filled does precipitation become available for filling other storage volumes. Water in canopy interception storage is held until it is removed by evaporation. Canopy interception has the same effect as the Initial Abstraction in *HEC-1*.

<u>Surface-interception storage</u>

Surface interception storage is the volume of water held in shallow surface depressions. Inflows to this storage come from precipitation not captured by canopy interception and in excess of the infiltration rate. Outflows from this storage can be due to infiltration and to evapotranspiration. Any water in surface depression storage at the beginning of the time step is available for infiltration. If the water available for infiltration exceeds the infiltration rate times the time step, surface interception storage is filled. Once the volume of surface interception is exceeded, the excess contributes to surface runoff.

For the purposes of this study, the canopy interception option was used to simulate *IA* for direct comparison of *HEC-HMS* model results with *HEC-1* and *FLO-2D*. The surface-interception storage was set to zero.

4.2.1.4 *FLO-2D* Approach

The *G&A* method is implemented in *FLO-2D* using an explicit form of the *G&A* equation developed in *Water and Sediment Routing from Complex Watersheds and Example Application to Surface Mining* (Fullerton, 1983). The form of the *G&A* equation used by Fullerton is:

$$\frac{\Delta F}{\gamma} - \ln\left(1 + \frac{\Delta F}{\gamma + F(t)}\right) = \frac{K_w}{\gamma}\Delta t$$

where:

ΔF = change in infiltration over the computation time step

K_w = *XKSAT*

γ = (*PSIF* + *Head*) * *DTHETA*

Head = incremental rainfall for the time step plus flow depth on the grid element

$F(t)$ = total infiltration at time t

Δt = computation time step

Fullerton developed an explicit equation for ΔF by using a power series expansion to approximate the logarithmic term in the above equation:

$$\Delta \mathrm{F} = \frac{-[2F(t) - K_w\Delta t] + [(2F(t) - K_w\Delta t)^2 + 8K_w\Delta t(\gamma + F(t))]^{0.5}}{2}$$

FLO-2D uses a global parameter named *TOL* that defines the minimum flow depth before rainfall excess or flow depth is exchanged with adjacent grids. This has a similar effect on the infiltration process as *IA* and is normally treated as the depression storage portion of *IA*. Setting the *IA* equal to only the sum of interception and evaporation enables the *TOL* value to represent the depression storage. Since rainfall excess for any given time step is usually very small, *TOL* needs to be small in order for rainfall excess to be accurately modeled. Therefore, when modeling rainfall-runoff with *FLO-2D*, the *FCDMC* requires that the *TOL* parameter be set to a small value, typically equal to the estimated depression storage or the smallest *IA* value in the model. The *FCDMC* typically uses a *TOL* setting of 0.004 feet, which is the smallest *IA* value for pavement and roof surfaces. The *TOL* value is then subtracted from the standard *IA* value and the result assigned as *IA* in the INFIL.DAT *FLO-2D* input data file. The desired total abstraction value should be equal to the sum of the assigned abstraction (interception) in the RAIN.DAT (global with or without infiltration) or INFIL.DAT (global and spatial variable *IA*) files plus the *TOL* depression storage. The minimum value of *TOL* that *FLO-2D* will accept is 0.001 feet.

The combined rainfall loss and transmission loss computation process followed by *FLO-2D* is as follows:

1. *IA* Not Satisfied. The initial abstraction is satisfied prior to rainfall infiltration calculations. Any infiltration that occurs while *IA* is being met is assumed to be included in *IA*. For each time step, incremental rainfall is summed until the total equals or exceeds *IA*.
2. *TOL* Not Satisfied. After *IA* is satisfied, *TOL* is checked against the incremental rainfall plus flow depth for the time step. If *TOL* is larger, then no infiltration is computed and that depth is set as the ponded depth. For the next time step, the comparison is made with *TOL* minus the ponded depth

from the previous time step. No infiltration is computed unless the rainfall for the time step plus the depth of incoming flow is greater than *TOL* minus the ponded depth.

3. Infiltration. *FLO-2D* computes the potential infiltration during the time step if the flow depth on the grid including the incremental rainfall depth is greater than *TOL*. If the potential infiltration is greater than the incremental rainfall plus flow depth, all depth during the time step is infiltrated including ponded depth. If the potential infiltration is less than the incremental rainfall plus flow depth, all potential infiltration is treated as infiltration and the difference between incremental rainfall and infiltration plus flow depth is accounted for as rainfall excess.
4. The addition of the head term allows *FLO-2D* to compute rainfall loss and transmission loss simultaneously.

4.2.1.5 Comparison of *FLO-2D* Results with *HEC-1* and *HEC-HMS*

<u>*HEC-1* Test Models</u>

A series of single basin *HEC-1* models were built, one for each of the eleven soil texture classes defined in Chapter 4 of the *DDM Hydrology*. The *FCDMC* standard 6-hour design storm was applied using a point rainfall depth of 5 inches. The *FCDMC* standard *G&A* parameter values were applied as listed in Table 4.1.

Table 4.1 Soil Texture Class G&A Parameters

Texture Class	*IA*, in	*PSIF*, in	*DTHETA*	*XKSAT*, in/hr
(1)	(2)	(3)	(4)	(5)
Loamy Sand and Sand	0.15	2.4	0.35	1.20
Sandy Loam	0.15	4.3	0.35	0.40
Loam	0.15	3.5	0.35	0.25
Silty Loam	0.15	6.6	0.40	0.15
Silt	0.15	7.5	0.35	0.10
Sandy Clay Loam	0.15	8.6	0.25	0.06
Clay Loam	0.15	8.2	0.25	0.04
Silty Clay Loam	0.15	10.8	0.30	0.04
Sandy Clay	0.15	9.4	0.20	0.02

Table 4.1 Soil Texture Class G&A Parameters				
Texture Class	***IA*, in**	***PSIF*, in**	***DTHETA***	***XKSAT*, in/hr**
(1)	**(2)**	**(3)**	**(4)**	**(5)**
Silty Clay	0.15	11.5	0.20	0.02
Clay	0.15	12.4	0.15	0.01

HEC-1 does not output rainfall infiltration and excess for each computation time step at a precision greater than two hundredths of an inch. In order to make a time step by time step comparison with *FLO-2D*, the FORTRAN source code for *HEC-1* was used in an independent computer program to compute infiltration and excess at a higher level of precision. The program is referred to herein as the *G&A Computation Program*. The *G&A Computation Program* was also used to test the addition of a head term in the *G&A* equation as set forth in *Applied Hydrology* (Chow, Maidment, & Mays, 1988). The program outputs *HEC-1* results for both with and without the head term. The results were checked against *HEC-1* to be sure that the total amount of infiltration and excess reported by *HEC-1* matches the *G&A Computation Program* results without the head term. After the *G&A Computation Program* results were found to match *HEC-1*, the with head results were checked against *FLO-2D* model results. Those results were then used to verify that the *FLO-2D* model meets the *FCDMC* defined standard based on *HEC-1* as described below under Assessment of Results.

***HEC-HMS* Test Model**

An *HEC-HMS* model was created using eleven sub-basins, one for each soil texture class. The same parameters applied in the *HEC-1* test models were used in the *HEC-HMS model*. The canopy-interception storage option was used instead of the surface-interception storage option in order to mimic the *HEC-1* and *FLO-2D* processes. The results were then used for comparison with *FLO-2D* and *HEC-1* as a confidence check on the rainfall loss computation method.

***FLO-2D* Test Models**

A separate *FLO-2D* model was built for each soil texture class listed in Table 4.1. Each model consisted of 81 grids with identical input data sets except for the *G&A* parameters. The grid elevations define a pyramid with Grid 41 being the highest point and all other grids being lower and on a uniform slope away from Grid 41. Rainfall was only applied to Grid 41. Each model was run with *TOL* set to 0.001 feet and 0.004 feet. The limiting infiltration depth value was set to 20.0 feet to ensure that the results would not be affected by a limiting control layer. Initial saturation was set to zero (0), final saturation was set to one (1), and *POROS* was set to zero (0).

FLO-2D does not output the infiltration values for every time step, only the total infiltration for the model. In order to adequately review the *FLO-2D* implementation, the *FLO-2D* FORTRAN source code for the rainfall losses subroutines was obtained from *FLO-2D* Software, Inc. This code was then reviewed and implemented in the *G&A Computation Program* discussed under HEC-1 Test Models and the total infiltration results checked against the values reported in the FPINFILTRATION.OUT *FLO-2D* output files to verify proper implementation. *FLO-2D* Software, Inc. also provided detailed infiltration output for the Sandy Loam texture class for *TOL* = 0.001 feet. This was used to check the results of the *G&A Computation Program*.

Assessment of Results

FLO-2D produces rainfall infiltration and excess results that closely match both *HEC-1* and *HEC-HMS*.

The *FLO-2D* results are compared with *HEC-1* and *HEC-HMS* in Table 4.2 and Table 4.3. The first things to understand are the comparisons made to verify that the *G&A Computation Program* reproduces the *HEC-1* and *FLO-2D* methods. There are three steps in that verification, using Table 4.2.

1. The first step was to verify that the *HEC-1* FORTRAN source code in the *G&A Computation Program* produces the same rainfall excess results as *HEC-1*. Since *HEC-1* does not include the head term in the G&A solution, the results from the *G&A Computation Program HEC-1* option are also based on no head term for this check. The results are compared in columns 2 and 3. Note that the results are identical to two decimal places, which is the level of precision reported by *HEC-1*.
2. The second step was to check the rainfall excess results between the *G&A Computation Program HEC-HMS* option and *HEC-HMS*. Comparing column 3 with 5, the results are identical to three decimal places as shown in column 7. The results of a time step by time step check also are identical to three decimal places.
3. The third step was to verify that the *G&A Computation Program FLO-2D* method computes the same total infiltration results as *FLO-2D*. Those results are shown in columns 9, 10, and 12. Note that the differences listed in column 12 are very small, 0.01 inch or less, except for the Sandy Loam and Loamy Sand texture classes. The largest difference, 0.0343 inches, is only 0.69% of the total rainfall. The *G&A Computation Program FLO-2D* method does not address *TOL* separately. Instead it handles *IA* in the same way that *HEC-1* does. The sum of *IA* and *TOL* was used for *IA* in the *G&A Computation Program*. No infiltration was computed until *IA* was satisfied. The *FLO-2D* model handles *TOL* as depression storage that can be drained completely depending on potential infiltration.

Also, *FLO-2D* uses a variable time step that is usually much smaller than 1 minute. It can compute the point in time where rainfall excess begins to occur more accurately than the *G&A Computation Program FLO-2D* solution and is using the actual flow depth plus incremental rainfall. The approximate explicit numerical solution by Fullerton may also be a part of the differences for the soils with large *XKSAT* values. These differences are minor. The *FLO-2D* method provides a more accurate depiction of the physical process and is accepted for *FCDMC* purposes.

The *G&A Computation Program* total rainfall excess results were checked between *HEC-1* (column 4) and *FLO-2D* (column 6). A *TOL* value of 0.001 feet was used in the *FLO-2D* models. Note that the differences between columns 4 and 6 listed in column 8 are very small, 0.01 inch or less, except for the Sandy Loam and Loamy Sand texture classes. The largest difference, 0.0433 inches, is only 0.87% of the total rainfall. These differences are attributed to how *FLO-2D* handles *TOL* and to differences between the Fullerton explicit solution of the *G&A* equation as opposed to the implicit solution applied by *HEC-1*.

Next, the *G&A Computation Program FLO-2D* method total rainfall infiltration results (column 9) were checked against *FLO-2D* (column 10). Note that the average of the differences between columns 9 and 10 listed in column 12 is less than 0.01 inch, or less than 0.2% of total rainfall. These differences are small enough that the results are acceptable to *FCDMC*.

Next, the *G&A Computation Program* solution results of the *HEC-1* method for rainfall excess with the head term applied (column 4) were compared with the independent computer results using the *FLO-2D* method (column 13). Note that the *G&A Computation Program* solution of the *FLO-2D* method for rainfall excess with the head term applied compares well with *HEC-1* because *TOL* and *IA* are being handled using the *HEC-1* approach. The differences between the two are listed in column 14 and the average difference is less than 0.01 inch.

Finally, the *G&A Computation Program* total rainfall excess results were checked between *HEC-1* and *FLO-2D* using a *TOL* value of 0.004 feet. This was done because a *TOL* of 0.004 feet is frequently used in *FCDMC* models to help reduce model run times and matches the smallest *FCDMC IA* value for pavement and roof surfaces. A value of 0.001 is more accurate, but slows the model down. It also results in negative values of *IA* when *TOL* is subtracted from *IA*. Those results are shown in Table 4.3. That comparison is made using columns 4, 6, and 8. Note that the differences listed in column 8 are larger than for a *TOL* of 0.001 feet. The finer grain soil texture classes vary from -0.02 to -0.09 inches, while the coarser grained soils are similar to the results from a *TOL* value of 0.001 feet. These differences are still relatively small (1.88% of total rainfall or less), and since the net effect is to increase rainfall excess, the differences are acceptable for *FCDMC* purposes.

Table 4.2 Comparison of Results: *FLO-2D* with *HEC-1* and *HEC-HMS* (*TOL* 0.001)

Head Term Status: On   LID: 20.00 feet

Texture Class	Rainfall Excess, inches						
	From *HEC-1*	*G&A Computation Program* *HEC-1* Method		*HEC-HMS*	*FLO-2D*	Differences	
						w/*HEC-HMS*	w/*FLO-2D*
	w/o Head	w/o Head	w/Head	w/o Head	w/Head	(3)-(5)	(4)-(6)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Loamy Sand and Sand	2.12	2.1243	2.1213	2.1242	2.0780	0.0001	0.0433
Sandy Loam	2.87	2.8667	2.8650	2.8665	2.8364	0.0002	0.0286
Loam	3.25	3.2475	3.2461	3.2474	3.2312	0.0001	0.0149
Silty Loam	3.24	3.2442	3.2433	3.2443	3.2300	-0.0001	0.0133
Silt	3.44	3.4413	3.4404	3.4411	3.4364	0.0002	0.0040
Sandy Clay Loam	3.73	3.7324	3.7318	3.7321	3.7292	0.0003	0.0026
Clay Loam	3.94	3.9426	3.9421	3.9425	3.9416	0.0001	0.0005
Silty Clay Loam	3.78	3.7755	3.7750	3.7755	3.7736	0.0000	0.0014
Sandy Clay	4.22	4.2188	4.2186	4.2201	4.2176	-0.0013	0.0010
Silty Clay	4.17	4.1673	4.1670	4.1662	4.1672	0.0011	-0.0002
Clay	4.40	4.3976	4.3975	4.397	4.3964	0.0006	0.0011
					Averages:	**0.0001**	**0.0101**

Texture Class	*FLO-2D* Infiltration, inches				*FLO-2D* Rainfall Excess, inches		Infiltration Penetration Depth, feet		
	G&A Computation Program	From *FLO-2D* FPINFILTRATION.OUT w/Head[1]		Difference w/*FLO-2D*	*G&A Computation Program*	Difference w/*HEC-1*	*FLO-2D* from INFIL_DEPTH.OUT	Check	Difference
	w/Head	inches	feet	(9)-(10)	w/Head	(4)-(13)			(15)-(16)
(1)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Loamy Sand and Sand	2.7377	2.7720	0.2310	-0.0343	2.1123	0.0090	0.66	0.6600	0.0000
Sandy Loam	1.9892	2.0136	0.1678	-0.0244	2.8608	0.0042	0.48	0.4794	0.0006
Loam	1.6069	1.6188	0.1349	-0.0119	3.2431	0.0030	0.39	0.3854	0.0046
Silty Loam	1.6084	1.6200	0.1350	-0.0116	3.2416	0.0017	0.34	0.3375	0.0025
Silt	1.4100	1.4136	0.1178	-0.0036	3.4400	0.0004	0.34	0.3366	0.0034
Sandy Clay Loam	1.1179	1.1208	0.0934	-0.0029	3.7321	-0.0002	0.37	0.3736	-0.0036
Clay Loam	0.9076	0.9084	0.0757	-0.0008	3.9424	-0.0003	0.30	0.3028	-0.0028
Silty Clay Loam	1.0747	1.0764	0.0897	-0.0017	3.7753	-0.0003	0.30	0.2990	0.0010
Sandy Clay	0.6310	0.6324	0.0527	-0.0014	4.2190	-0.0004	0.26	0.2635	-0.0035
Silty Clay	0.6826	0.6828	0.0569	-0.0002	4.1674	-0.0004	0.28	0.2845	-0.0045
Clay	0.4521	0.4536	0.0378	-0.0015	4.3979	-0.0004	0.25	0.2520	-0.0020
			Averages:	**-0.0086**		**0.0015**			**-0.0004**

[1] Based on TOL = 0.001 ft

(blue) Generated externally using *HEC-HMS* and hard-coded in spreadsheet
(yellow) Generated externally using *FLO-2D* and hard-coded in spreadsheet
(orange) Generated externally using *HEC-1* and hard-coded in spreadsheet

Table 4.3 Comparison of Results: *FLO-2D* with *HEC-1* and *HEC-HMS* (*TOL* 0.004)

Head Term Status: On **LID:** 20.00 feet

Texture Class	Rainfall Excess, inches						
	From *HEC-1*	*G&A Computation Program* *HEC-1* Method		*HEC-HMS*	*FLO-2D*	Differences	
	w/o Head	w/o Head	w/Head	w/o Head	w/Head	w/*HEC-HMS* (3)-(5)	w/*FLO-2D* (4)-(6)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Loamy Sand and Sand	2.12	2.1243	2.1213	2.1242	2.0900	0.0001	0.0313
Sandy Loam	2.87	2.8667	2.8650	2.8665	2.8508	0.0002	0.0142
Loam	3.25	3.2475	3.2461	3.2474	3.2432	0.0001	0.0029
Silty Loam	3.24	3.2442	3.2433	3.2443	3.2396	-0.0001	0.0037
Silt	3.44	3.4413	3.4404	3.4411	3.4592	0.0002	-0.0188
Sandy Clay Loam	3.73	3.7324	3.7318	3.7321	3.7532	0.0003	-0.0214
Clay Loam	3.94	3.9426	3.9421	3.9425	4.0172	0.0001	-0.0751
Silty Clay Loam	3.78	3.7755	3.7750	3.7755	3.8276	0.0000	-0.0526
Sandy Clay	4.22	4.2188	4.2186	4.2201	4.3124	-0.0013	-0.0938
Silty Clay	4.17	4.1673	4.1670	4.1662	4.2524	0.0011	-0.0854
Clay	4.40	4.3976	4.3975	4.397	4.4720	0.0006	-0.0745
					Averages:	**0.0001**	**-0.0336**

Texture Class	*FLO-2D* Infiltration, inches				*FLO-2D* Rainfall Excess, inches		Infiltration Penetration Depth, feet		
	G&A Computation Program	From *FLO-2D* FPINFILTRATION.OUT w/Head[1]		Difference w/*FLO-2D*	*G&A Computation Program*	Difference w/*HEC-1*	*FLO-2D* from INFIL_DEPTH.OUT	Check	Difference
	w/Head	inches	feet	(9)-(10)	w/Head	(4)-(13)			(15)-(16)
(1)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Loamy Sand and Sand	2.7377	2.7600	0.2300	-0.0223	2.1123	0.0090	0.66	0.6571	0.0029
Sandy Loam	1.9892	1.9992	0.1666	-0.0100	2.8608	0.0042	0.48	0.4760	0.0040
Loam	1.6069	1.6068	0.1339	0.0001	3.2431	0.0030	0.38	0.3826	-0.0026
Silty Loam	1.6084	1.6104	0.1342	-0.0020	3.2416	0.0017	0.34	0.3355	0.0045
Silt	1.4100	1.3908	0.1159	0.0192	3.4400	0.0004	0.33	0.3311	-0.0011
Sandy Clay Loam	1.1179	1.0968	0.0914	0.0211	3.7321	-0.0002	0.37	0.3656	0.0044
Clay Loam	0.9076	0.8328	0.0694	0.0748	3.9424	-0.0003	0.28	0.2776	0.0024
Silty Clay Loam	1.0747	1.0224	0.0852	0.0523	3.7753	-0.0003	0.28	0.2840	-0.0040
Sandy Clay	0.6310	0.5376	0.0448	0.0934	4.2190	-0.0004	0.22	0.2240	-0.0040
Silty Clay	0.6826	0.5976	0.0498	0.0850	4.1674	-0.0004	0.25	0.2490	0.0010
Clay	0.4521	0.3780	0.0315	0.0741	4.3979	-0.0004	0.21	0.2100	0.0000
			Averages:	**0.0351**		**0.0015**			**0.0007**

[1] Based on TOL = 0.004 ft

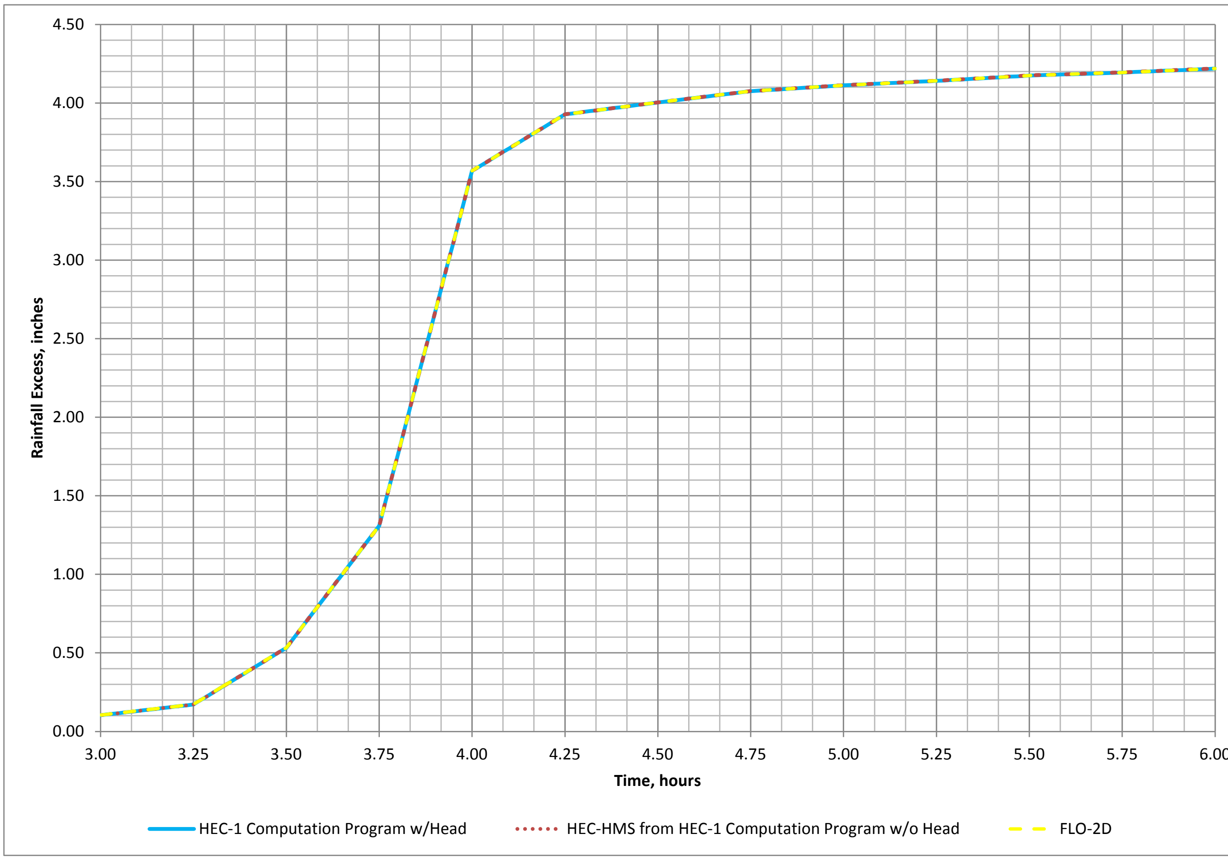
Generated externally using *HEC-HMS* and hard-coded in spreadsheet

Generated externally using *FLO-2D* and hard-coded in spreadsheet

Generated externally using *HEC-1* and hard-coded in spreadsheet

Table 4.2 also contains information on the depth of penetration into the soil matrix by infiltrated water. That information is listed in columns 15 through 17. The data in column 15 is taken from the INFIL_DEPTH.OUT *FLO-2D* output file. The penetration depth is computed by dividing the infiltration depth by *DTHETA*. The *FLO-2D* model is producing the expected results, verified by comparison with the *G&A Computation Program* results listed in column 16. An example graph of rainfall excess for the entire storm duration for a sandy clay texture class and *TOL* set to 0.001 feet is shown on Figure 4.1. Note that the rainfall excess results from the *G&A Computation Program HEC-1*, *HEC-HMS*, and *FLO-2D* methods shown on Figure 4.1 are indistinguishable from each other.

Figure 4.1 Comparison of Rainfall Excess Results for a Sandy Clay (*TOL* 0.001)



4.2.1.6 Head Term for Large Flow Depths

As can be seen from evaluation of the data in Table 4.2, the head term is negligible for small flow depths. However, for larger flow depths, it can be significant. To test the accuracy of the head term applied in the *G&A* implementation in *FLO-2D*, the simple pyramid model described in Section 4.2.1.5 was modified to be a level pool model in order to force a ponded depth of 10 feet without exchange of flow with adjacent grids as follows:

1. The elevation of Grid 41 was left unchanged at elevation 1,000.00.
2. The elevation of all other grids was set to elevation 1,020.00.
3. Rain was turned off.
4. An INFLOW.DAT file was added and an initial ponded water surface set at elevation 1010.00.
5. *TOL* was set to 0.001 feet.
6. Limiting infiltration was left unchanged at 20.0 feet.

This has the effect of forcing an initial ponded depth of 10 feet without any other inflow or rainfall. Flow cannot exit Grid 41 except by infiltration.

All eleven texture classes were run with these changed conditions. The *G&A Computation Program* was used to mimic the same conditions for the *HEC-1* and *FLO-2D* code options. The results are compared in Table 4.4. First note that the *G&A Computation Program* infiltration results are almost identical with the *FLO-2D* results. Refer to columns 9, 10, and 12. This is verification that the *G&A Computation Program* is reproducing the *FLO-2D* results. Second, referring to columns 4, 6 and 8, the *FLO-2D* rainfall excess results compare favorably with the *G&A Computation Program HEC-1* results. The largest difference is for the Loamy Sand texture class, 0.87 inches out of 28 inches of infiltration (about a 3.1% difference) over the 6-hour time period.

Since the *G&A Computation Program FLO-2D* results check with *FLO-2D*, the differences in the coarser grained soil texture classes are attributable to the Fullerton explicit solution of the *G&A* equation as opposed to the implicit solution applied by *HEC-1*. Since the differences are relatively small and the result is slightly more rainfall excess on the surface, the results are acceptable for *FCDMC* purposes.

The same check was done using *TOL* set to 0.004 feet. The results are identical to those above for *TOL* set to 0.001 feet.

Table 4.4 Comparison of Results: *FLO-2D* Ponded Depth Results with *HEC-1* (*TOL* 0.001)

Head Term Status: On **Starting Head:** 10 feet **LID:** 20.00 feet

	Rainfall Excess, inches						
	From	*G&A Computation Program*				Differences	
	HEC-1	*HEC-1 Method*		*HEC-HMS*	*FLO-2D*	w/*HEC-HMS*	w/*FLO-2D*
Texture Class	w/o Head	w/o Head	w/Head	w/o Head	w/Head	(3)-(5)	(4)-(6)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Loamy Sand and Sand	n/a	n/a	90.9277	n/a	91.7964	n/a	-0.8687
Sandy Loam	n/a	n/a	104.0563	n/a	104.3400	n/a	-0.2837
Loam	n/a	n/a	107.6273	n/a	107.8032	n/a	-0.1759
Silty Loam	n/a	n/a	109.8202	n/a	109.9392	n/a	-0.1190
Silt	n/a	n/a	112.2012	n/a	112.2696	n/a	-0.0684
Sandy Clay Loam	n/a	n/a	114.8284	n/a	114.8568	n/a	-0.0284
Clay Loam	n/a	n/a	115.7881	n/a	115.8072	n/a	-0.0191
Silty Clay Loam	n/a	n/a	115.3747	n/a	115.3968	n/a	-0.0221
Sandy Clay	n/a	n/a	117.2863	n/a	117.2928	n/a	-0.0065
Silty Clay	n/a	n/a	117.2662	n/a	117.2736	n/a	-0.0074
Clay	n/a	n/a	118.2697	n/a	118.2720	n/a	-0.0023
					Averages:		**-0.1456**

	FLO-2D Infiltration, inches				*FLO-2D* Rainfall Excess, inches		Infiltration Penetration Depth, feet		
	G&A Computation Program	From *FLO-2D* FPINFILTRATION.OUT w/Head[1]		Difference w/*FLO-2D*	*G&A Computation Program*	Difference w/*HEC-1*	*FLO-2D* from		Difference
Texture Class	w/Head	inches	feet	(9)-(10)	w/Head	(4)-(13)	INFIL_DEPTH.OUT	Check	(15)-(16)
(1)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Loamy Sand and Sand	28.0546	28.0536	2.3378	0.0010	91.7954	-0.8678	6.68	6.6794	0.0006
Sandy Loam	15.5102	15.5100	1.2925	0.0002	104.3398	-0.2836	3.69	3.6929	-0.0029
Loam	12.0473	12.0468	1.0039	0.0005	107.8027	-0.1754	2.87	2.8683	0.0017
Silty Loam	9.9112	9.9108	0.8259	0.0004	109.9388	-0.1186	2.06	2.0648	-0.0047
Silt	7.5804	7.5804	0.6317	0.0000	112.2696	-0.0684	1.80	1.8049	-0.0049
Sandy Clay Loam	4.9930	4.9932	0.4161	-0.0002	114.8570	-0.0286	1.66	1.6644	-0.0044
Clay Loam	4.0432	4.0428	0.3369	0.0004	115.8068	-0.0186	1.35	1.3476	0.0024
Silty Clay Loam	4.4528	4.4532	0.3711	-0.0004	115.3972	-0.0225	1.24	1.2370	0.0030
Sandy Clay	2.5567	2.5572	0.2131	-0.0005	117.2933	-0.0070	1.07	1.0655	0.0045
Silty Clay	2.5768	2.5764	0.2147	0.0004	117.2732	-0.0070	1.07	1.0735	-0.0035
Clay	1.5780	1.5780	0.1315	0.0000	118.2720	-0.0023	0.88	0.8767	0.0033
			Averages:	**0.0002**		**-0.1454**			**-0.0004**

[1] Based on TOL = 0.001 ft

(blue) Generated externally using *HEC-HMS* and hard-coded in spreadsheet
(yellow) Generated externally using *FLO-2D* and hard-coded in spreadsheet
(orange) Generated externally using *HEC-1* and hard-coded in spreadsheet

4.2.1.7 1D Channel Infiltration

General

The *1D* channel option includes the ability to model rainfall and transmission losses as described in Section 2.3.3.5. In order to verify that the infiltration calculations are done correctly, the flume model described in Section 4.2.2.3 was modified to simulate infiltration. Three inflow scenarios were created:

1. A uniform flow rate of 1,500 cfs was simulated for a period of 10 hours.
2. A standard 6-hour design storm using *DDM Hydrology* pattern 1 and a total rainfall depth of 5 inches without inflow.
3. A combination of Scenarios 1 and 2: both inflow and rainfall.

Models were created for testing as follows. The model directory name used for each is shown in parenthesis, typical.

Scenario 1: Inflow Only

A. Infiltration by reach, prismatic (Inflow_Reach_Infil_Prismatic_NoLID)
B. Infiltration by reach, cross sections (Inflow_Reach_Infil_XSEC_NoLID)
C. Infiltration by element, cross sections (Inflow_Element_Infil_XSEC_NoLID)

Scenario 2: Rainfall Only

A. Infiltration by reach, prismatic (Rainfall_Reach_Infil_Prismatic_NoLID)
B. Infiltration by reach, cross sections (Rainfall_Reach_Infil_XSEC_NoLID)
C. Infiltration by element, cross sections (Rainfall_Element_Infil_XSEC_NoLID)

Scenario 3: Inflow and Rainfall

A. Infiltration by reach, prismatic (Rainfall_Inflow_Reach_Infil_Prismatic_NoLID)
B. Infiltration by reach, cross sections (Rainfall_Inflow_Reach_Infil_XSEC_NoLID)

An infiltration by element was not needed for Scenario 3 as described in Scenario 3A: Rainfall and Inflow Model - Infiltration by reach – Prismatic. Channel infiltration was modeled using the *G&A* method. The *FLO-2D* model applies a default value for *PSIF* of 0.00 under the assumption that most channel soils are coarse grained and that the flow depth is so much greater than the capillary suction that *PSIF* can be neglected. The *DTHETA* parameter is applied uniformly for all channel reaches using the global values. With the *POROS* parameter set to zero, the difference between *SATF* and *SATI* is the *DTHETA* value applied by the model.

In order to verify the *FLO-2D* model results, the *G&A Computation Program* was used to compute infiltration by applying the *G&A* solution scheme used within *FLO-2D*. The models are described in the following sections, and the results presented and discussed under 1D Channel Infiltration Test Results.

Scenario 1A: Inflow Only Model - Infiltration by reach – Prismatic

This example is based on a uniform rectangular flume modeled using the following:

1. *FLO-2D* prismatic *1D* rectangular channel, base width = 110 ft, length = 950 ft. The actual channel length is 960 feet, but infiltration calculations are not performed by *FLO-2D* for the outflow element.
2. Slope = 0.002 ft/ft
3. *n* = 0.040
4. *TOL* = 0.10 ft (for inflow test models), 0.001 ft (for rainfall test models)
5. *IA* = 0.138 in
6. *XKSAT* = 0.40 in/hr (Sandy Loam)
7. *DTHETA* = 0.35
8. Inflow: 0 to 1,500 cfs in 2 hrs; 1,500 cfs for 8 hrs; total duration = 10 hrs
9. Duration of uniform flow used in the *G&A Computation Program* was 10 hours.
10. Infiltration modeled by applying the above *G&A* parameters uniformly over the entire reach.
11. Limiting infiltration depth option turned off.

This test model simulates a uniform flow depth over time except for the initial two hours when flow is increased from 0 to 1,500 cfs. The uniform depth of flow was taken from the HYCHAN.OUT file and is the average of flow depth for all cross sections for every time step. The *FLO-2D 1D* channel infiltration volume of 2.81 ac-ft for the run was taken from the CHVOLUME.OUT and SUMMARY.OUT files and is reported in Table 4.5 under *1D* Channel Infiltration Test Results. The verification unit infiltration volume in inches from the *G&A Computation Program* was converted to total infiltration in ac-ft as follows:

$$V_i = \frac{L_c * W_c * \frac{I_m}{12}}{43{,}560}$$

= (950*116.618*13.363/12)/43560 = 2.832 ac-ft

where:

V_i = Total computed infiltration volume in ac-ft

L_c = Length of channel = 95 grids*10 feet = 950 feet

W_c = Wetted perimeter of channel width = 11 grids*10 feet+2*D_a = 11*10+2*3.309 = 116.618 feet

D_a = Average uniform flow depth = 3.309 feet (computed from *FLO-2D* results)

I_m = Infiltration volume computed by the *G&A Computation Program* = 13.363 inches

The results are discussed below under 1D Channel Infiltration Test Results.

Scenario 1B: Inflow Only Model - Infiltration by reach – Cross Sections

This example is identical to Scenario 1A except that natural ground cross sections were used to define the rectangular channel. The results of this test were identical to Scenario 1A and are listed in Table 4.5 under *1D* Channel Infiltration Test Results.

Scenario 1C: Inflow Only Model - Infiltration by element – Cross Sections

This example uses the same geometry and parameters as Scenario 1B except that infiltration is spatially-varied, computed by channel element. This option cannot be used with a prismatic channel. The first upstream 47 cross sections were set to an *XKSAT* value of 0.40 in/hr (Sandy Loam). The remaining 49 cross sections were set to an *XKSAT* value of 0.01 in/hr (Clay). The total infiltration computed by *FLO-2D* was 1.58 ac-ft. This value was verified using the same approach described for Scenario 1A. The calculations were done separately for the two reaches of different *XKSAT* values and then totaled.

$$V_i = \frac{L_c * W_c * \frac{I_m}{12}}{43{,}560}$$

= (470*117.354*13.0694/12)/43560 = 1.379 ac-ft for XKSAT = 0.40 in/hr

= (490*117.354*1.7278/12)/43560 = 0.190 ac-ft for XKSAT = 0.01 in/hr

V_i Total = 1.379+0.190 = 1.569 ac-ft

where:

V_i = Total computed infiltration volume in ac-ft

L_c = Length of channel = 95 grids*10 feet = 950 feet

W_c = Wetted perimeter of channel width = 110+2*3.677 = 117.354 feet

D_a = Average uniform flow depth = 3.677 feet (computed from *FLO-2D* results)

I_m = Infiltration volume computed by the *G&A Computation Program* = 13.0694 inches for XKSAT = 0.40 in/hr

I_m = Infiltration volume computed by the *G&A Computation Program* = 1.7278 inches for XKSAT = 0.01 in/hr

The results are discussed below under 1D Channel Infiltration Test Results.

Scenario 2A: Rainfall Only Model - Infiltration by reach – Prismatic

This example uses the same geometry and parameters as Scenario 1A with the following exceptions:

1. Rainfall instead of runoff. A 6-hour design storm of 4.0 inches using a Type 1 rainfall distribution was used.
2. Channel base width is 15 feet
3. *TOL* was set to 0.001 feet.

The results from this model yielded a total infiltration volume of 0.08 ac-ft. It is very difficult to do a verification of the model infiltration for this scenario because an individual cross section cannot be isolated. The reach is experiencing rainfall-runoff and accumulated flow depth over time. The best that can be done is a reasonableness check. To accomplish this, the *FLO-2D* results (HYCHAN.OUT) were analyzed to estimate an average flow depth over the length of channel for all time steps. The analysis yielded an estimated average flow depth of 0.079 feet. This depth was then applied in the *G&A Computation Program* as an initial ponded depth with the head held constant over time. The *G&A Computation Program* yielded an average infiltration depth of 2.8957 inches over the channel reach. The total infiltration volume was computed using the same approach applied for Scenario 1A. The verification unit infiltration volume in inches from the *G&A Computation Program* was converted to total infiltration in ac-ft as follows:

$$V_i = \frac{L_c * W_c * \frac{I_m}{12}}{43{,}560}$$

= (950*15.158*2.8957/12)/43560 = 0.080 ac-ft

where:

V_i = Total computed infiltration volume in ac-ft

L_c = Length of channel = 95 grids*10 feet = 950 feet

W_c = Wetted perimeter of channel width = 15 feet+2*D_a = 15+2*0.079 = 15.158 feet

D_a = Average uniform flow depth = 0.079 feet (computed from *FLO-2D* results)

I_m = Infiltration volume computed by the *G&A Computation Program* = 2.8957 inches

The results are discussed below under 1D Channel Infiltration Test Results.

Scenario 2B: Rainfall Only Model - Infiltration by reach – Cross Sections

This example uses the same geometry and parameters as Scenario 2A except that cross sections were used to define the uniform rectangular channel. The results from this model yielded a total infiltration volume of 0.06 ac-ft. It is very difficult to do a verification of the model infiltration for this scenario because an individual cross section cannot be isolated. The reach is experiencing rainfall-runoff and accumulated flow depth over time. The best that can be done is a reasonableness check. To accomplish this, the *FLO-2D* results (HYCHAN.OUT) were analyzed to estimate an average flow depth over the length of channel for all time steps. The analysis yielded an estimated average flow depth of 0.177 feet.

Note that this flow depth is greater than was computed for Scenario 2A using the prismatic channel option. *FLO-2D* generates a rating table of hydraulic parameters prior to running the model. *FLO-2D* then performs a regression analysis of this data to enable very fast computation of the relationship between depth, area, and wetted perimeter. This is extremely efficient for non-uniform natural channels, but is not necessarily as effective for a prismatic cross section defined using a minimum number of points. For very small flow depths in relatively wide channels, error can be introduced through this process. In the case of this example where flow depth is very small, the regression equations do not field flow over the entire width of the rectangular channel. Therefore, the wetted perimeter is less than the bottom width, and the computed flow depth is higher than it should be. The user should keep this in mind and use the prismatic channel option especially when the channel is relatively large in relation to the flow depth. For this example, the average wetted perimeter for all time steps over the entire length of reach was also computed. The average wetted perimeter computed by *FLO-2D* was 6.34 feet.

The average depth was then applied in the *G&A Computation Program* as an initial ponded depth with the head held constant over time. The *G&A Computation Program* yielded an average infiltration depth of 3.7361 inches over the channel reach. The total infiltration volume was computed using the same approach applied for Scenario 2A. The verification unit infiltration volume in inches from the *G&A Computation Program* was converted to total infiltration in ac-ft as follows:

$$V_i = \frac{L_c * W_c * \frac{I_m}{12}}{43{,}560}$$

= (950*6.343*3.7361/12)/43560 = 0.043 ac-ft

where:

V_i = Total computed infiltration volume in ac-ft

L_c = Length of channel = 95 grids*10 feet = 950 feet

W_c = Wetted perimeter of channel width = 6.343 feet (from FLO-2D results)

D_a = Average uniform flow depth = 0.177 feet (computed from *FLO-2D* results)

I_m = Infiltration volume computed by the *G&A Computation Program* = 3.7361 inches

The results are discussed below under 1D Channel Infiltration Test Results.

Scenario 2C: Rainfall Only Model - Infiltration by element – Cross Sections

This example uses the same geometry and parameters as Scenario 2B except that infiltration is spatially-varied, computed by channel element. This option cannot be used with a prismatic channel. The first upstream 47 cross sections were set to an *XKSAT* value of 0.40 in/hr (Sandy Loam). The remaining 49 cross sections were set to an *XKSAT* value of 0.01 in/hr (Clay). The total infiltration computed by *FLO-2D* was 0.03 ac-ft. This value was verified using the same approach described for Scenario 2B. The calculations were done separately for the two reaches of different *XKSAT* values and then totaled. The verification unit infiltration volumes in inches from the *G&A Computation Program* were converted to total infiltration in ac-ft as follows:

$$V_i = \frac{L_c * W_c * \frac{I_m}{12}}{43{,}560}$$

= (470*6.41*3.7493/12)/43560 = 0.022 ac-ft for XKSAT = 0.40 in/hr

= (490*6.65*0.3515/12)/43560 = 0.002 ac-ft for XKSAT = 0.01 in/hr

V_iTotal = 0.022+0.002 = 0.024 ac-ft

where:

V_i = Total computed infiltration volume in ac-ft

L_c = Length of channel = 95 grids*10 feet = 950 feet

W_c = Wetted perimeter of channel width = 6.41 feet (from FLO-2D results for XKSAT = 0.40 in/hr)

W_c = Wetted perimeter of channel width = 6.65 feet (from FLO-2D results for XKSAT = 0.01 in/hr)

D_a = Average uniform flow depth = 0.180 feet (computed from *FLO-2D* results for XKSAT = 0.40 in/hr)

D_a = Average uniform flow depth = 0.191 feet (computed from *FLO-2D* results for XKSAT = 0.01 in/hr)

I_m = Infiltration volume computed by the *G&A Computation Program* = 3.7493 inches for XKSAT = 0.40 in/hr

I_m = Infiltration volume computed by the *G&A Computation Program* = 0.3515 inches for XKSAT = 0.01 in/hr

The results are discussed below under 1D Channel Infiltration Test Results.

Scenario 3A: Rainfall and Inflow Model - Infiltration by reach – Prismatic

Scenarios 3A and 3B were done in order to check that infiltration computations are handled appropriately when both rainfall and external inflow are imposed at the same time. A test case for element by element infiltration was not done for this scenario because 3A and 3B in combination with Scenarios 1 and 2 make it irrelevant. This example uses the same geometry and parameters as Scenario 2A with the following exception:

1. Rainfall is combined with a uniform inflow rate of 25 cfs. A uniform rainfall of 6.0 inches every 0.1 hours is applied for a total depth of 144 inches. This was done to exaggerate the rainfall effects, which are often minimal compared with a uniform inflow when a normal design storm rainfall is applied.

The results from this model yielded a total channel infiltration volume of 0.25 ac-ft. The *FLO-2D* results (HYCHAN.OUT) were analyzed to estimate an average flow depth over the length of channel for all time steps. The analysis yielded an estimated average flow depth of 1.719 feet. This depth was then applied in the *G&A Computation Program* as an initial ponded depth with the head held constant over time. The *G&A Computation Program* yielded an average infiltration depth of 7.5843 inches over the channel reach. The total infiltration volume was computed using the same approach applied for Scenario 2A. The verification unit infiltration volume in inches from the *G&A Computation Program* was converted to total infiltration in ac-ft as follows:

$$V_i = \frac{L_c * W_c * \frac{I_m}{12}}{43{,}560}$$

= (950*18.437*7.5843/12)/43560 = 0.254 ac-ft

where:

V_i = Total computed infiltration volume in ac-ft

L_c = Length of channel = 95 grids*10 feet = 950 feet

W_c = Wetted perimeter of channel width = 15 feet+2*D_a = 15+2*1.719 = 18.437 feet

D_a = Average uniform flow depth = 1.719 feet (computed from *FLO-2D* results)

I_m = Infiltration volume computed by the *G&A Computation Program* = 7.5843 inches

The results are discussed below under 1D Channel Infiltration Test Results.

Scenario 3B: Rainfall and Inflow Model - Infiltration by reach – Cross Sections

This example uses the same geometry and parameters as Scenario 3A except that cross sections were used to define the uniform rectangular channel. The results from this model yielded a total infiltration volume of 0.26 ac-ft. The *FLO-2D* results (HYCHAN.OUT) were analyzed to estimate an average flow depth over the length of channel for all time steps. The analysis yielded an estimated average flow depth of 1.754 feet.

Note that this flow depth is slightly greater (0.035 feet) than was computed for Scenario 3A using the prismatic channel option. This minimal difference is due to the hydraulic parameter rating table (refer to Section 7.11.4.2). The average depth was then applied in the *G&A Computation Program* as an initial ponded depth with the head held constant over time. The *G&A Computation Program* yielded an average infiltration depth of 7.6432 inches over the channel reach. The total infiltration volume was computed using the same approach applied for Scenario 3A. The verification unit infiltration volume in inches from the *G&A Computation Program* was converted to total infiltration in ac-ft as follows:

$$V_i = \frac{L_c * W_c * \frac{I_m}{12}}{43{,}560}$$

= (950*18.508*7.6432/12)/43560 = 0.257 ac-ft

where:

V_i = Total computed infiltration volume in ac-ft

L_c = Length of channel = 95 grids*10 feet = 950 feet

W_c = Wetted perimeter of channel width = 15+2*1.754 = 18.508 feet (from *FLO-2D* results)

D_a = Average uniform flow depth = 1.754 feet (computed from *FLO-2D* results)

I_m = Infiltration volume computed by the *G&A Computation Program* = 7.6432 inches

The results are discussed below under 1D Channel Infiltration Test Results.

***1D* Channel Infiltration Test Results**

The channel infiltration testing results are summarized in Table 4.5. The differences listed in column 5 include the difference in ac-ft and the percent difference. Scenarios 2B and 2C have high percentage differences (20-28%), but the magnitude of infiltration is so small (<0.02 ac-ft), this is not a concern. For all test cases, the differences in computed infiltration volume between *FLO-2D* and the independent verification calculations is minimal and within acceptable limits. The *FLO-2D 1D* channel infiltration loss method is acceptable for *FCDMC* purposes.

Table 4.5 *1D* Channel Infiltration Testing Results				
Scenario	Infiltration Volume			
	***FLO-2D*, ac-ft**	***G&A* Computation Program**		**Difference, ac-ft (%)**
		inches	**ac-ft**	
(1)	**(2)**	**(3)**	**(4)**	**(5)**
1A: Inflow only, infiltration by reach, prismatic section	2.81	13.0638	2.786	0.024 (0.86)
1B: Inflow only, infiltration by reach, cross sections	2.81	13.0638	2.786	0.024 (0.86)
1C: Inflow only, infiltration by element, cross sections	1.58	13.0694 (0.40 *XKSAT*) 1.7278 (0.01 *XKSAT*)	1.569	0.011 (0.70)
2A: Rainfall only, infiltration by reach, prismatic section	0.08	2.8957	0.080	0.000 (0.00)
2B: Rainfall only, infiltration by reach, cross sections	0.06	3.7361	0.043	0.017 (28)
2C: Rainfall only, infiltration by element, cross sections	0.030	3.7493 (0.40 *XKSAT*) 0.3515 (0.01 *XKSAT*)	0.024	0.006 (20)
3A: Rainfall and inflow, infiltration by reach, prismatic section	0.25	7.5843	0.254	-0.004 (-1.6)
3B: Rainfall and inflow, infiltration by reach, cross sections	0.26	7.6432	0.257	0.003 (1.15)

4.2.1.8 Multiple Channels Infiltration

Still under development, not included in this edition. Not necessary for FEMA approval.

4.2.1.9 Limiting Infiltration Depth

Floodplain Grids

The limiting infiltration depth (*LID*) option was tested using the *FLO-2D* model developed for testing the head term for large flow depths described in Section 4.2.1.6. Under that model, without a limiting depth, the infiltration ranges from a minimum of 0.88 feet to 6.7 feet. For this test, the *LID* assignment (global and spatially varied) was set to 0.5 feet for all eleven texture class models. The results are compared in columns 15 through 17 of Table 4.6.

The depth of penetration of infiltrated water is equal to the infiltration depth divided by *DTHETA*. The depth of penetration calculated by *FLO-2D* is written to the INFIL_DEPTH.OUT file (values are listed in column 15). The penetration depth computed using the *G&A Computation Program* is listed in column 16. The differences listed in column 17 are all less than 0.01 feet. Based on this test, the *FLO-2D LID* option works well for floodplain grid infiltration.

***1D* Channels**

The *1D* channel *LID* tests were done using only inflow (no rainfall). Two scenarios were considered:

Scenario 1. Infiltration by reach (1_Inflow_Reach_Infil_Prismatic_LID).

Scenario 2. Infiltration by element (2_Inflow_Reach_Infil_XSEC_LID).

Scenario 1: Inflow Only Model - Infiltration by reach – Prismatic - *LID*

This example is based on a uniform rectangular flume modeled using the following:

1. *FLO-2D* prismatic *1D* rectangular channel, base width = 110 ft, length = 950 ft. The actual channel length is 960 feet, but infiltration calculations are not performed by *FLO-2D* for the outflow element.
2. Slope = 0.002 ft/ft
3. *n* = 0.040
4. *TOL* = 0.10 ft
5. *IA* = 0.138 in
6. *XKSAT* = 0.40 in/hr (Sandy Loam)
7. *DTHETA* = 0.35
8. Inflow: 0 to 1,500 cfs in 2 hrs; 1,500 cfs for 8 hrs; total duration = 10 hrs
9. Duration of uniform flow used in the *G&A Computation Program* was 9 hours.
10. Infiltration modeled by applying the above *G&A* parameters uniformly over the entire reach.
11. Limiting infiltration depth option turned on and set to 0.5 feet.

Table 4.6 Comparison of Results: FLO-2D *LID* Results with HEC-1 (*TOL* 0.001)

Head Term Status: On Starting Head: 10 feet LID: 0.50 feet

Texture Class	Rainfall Excess, inches: From *HEC-1* w/o Head	G&A Computation Program *HEC-1* Method w/o Head	G&A Computation Program *HEC-1* Method w/Head	*HEC-HMS* w/o Head	*FLO-2D* w/Head	Differences w/*HEC-HMS* (3)-(5)	Differences w/*FLO-2D* (4)-(6)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Loamy Sand and Sand	n/a	n/a	117.5431	n/a	117.5484	n/a	-0.0053
Sandy Loam	n/a	n/a	117.6634	n/a	117.7392	n/a	-0.0758
Loam	n/a	n/a	117.6543	n/a	117.7476	n/a	-0.0933
Silty Loam	n/a	n/a	117.4049	n/a	117.4392	n/a	-0.0343
Silt	n/a	n/a	117.7095	n/a	117.7332	n/a	-0.0237
Sandy Clay Loam	n/a	n/a	118.3294	n/a	118.3428	n/a	-0.0134
Clay Loam	n/a	n/a	118.3392	n/a	118.3488	n/a	-0.0096
Silty Clay Loam	n/a	n/a	118.0257	n/a	118.0440	n/a	-0.0183
Sandy Clay	n/a	n/a	118.6370	n/a	118.6500	n/a	-0.0130
Silty Clay	n/a	n/a	118.6424	n/a	118.6476	n/a	-0.0052
Clay	n/a	n/a	118.9467	n/a	118.9488	n/a	-0.0021
					Averages:		-0.0267

Texture Class	FLO-2D Infiltration, inches: G&A Computation Program w/Head	From *FLO-2D* FPINFILTRATION.OUT w/Head[1] inches	From *FLO-2D* FPINFILTRATION.OUT w/Head[1] feet	Difference w/*FLO-2D*	FLO-2D Rainfall Excess, inches: G&A Computation Program w/Head	Difference w/*HEC-1*	Infiltration Penetration Depth, feet: *FLO-2D* from INFIL_DEPTH.OUT	Check	Difference
				(9)-(10)		(4)-(13)		(10)/*DTHETA*/12	(15)-(16)
(1)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Loamy Sand and Sand	2.2956	2.3016	0.1918	-0.0060	117.7044	-0.1613	0.55	0.5480	0.0020
Sandy Loam	2.1782	2.1108	0.1759	0.0674	117.8218	-0.1584	0.50	0.5026	-0.0026
Loam	2.1014	2.1024	0.1752	-0.0010	117.8986	-0.2444	0.50	0.5006	-0.0006
Silty Loam	2.4367	2.4108	0.2009	0.0259	117.5633	-0.1584	0.50	0.5023	-0.0022
Silt	2.1344	2.1168	0.1764	0.0176	117.8656	-0.1561	0.50	0.5040	-0.0040
Sandy Clay Loam	1.5178	1.5072	0.1256	0.0106	118.4822	-0.1528	0.50	0.5024	-0.0024
Clay Loam	1.5081	1.5012	0.1251	0.0069	118.4919	-0.1526	0.50	0.5004	-0.0004
Silty Clay Loam	1.8056	1.8060	0.1505	-0.0004	118.1944	-0.1687	0.50	0.5017	-0.0017
Sandy Clay	1.2041	1.2000	0.1000	0.0041	118.7959	-0.1589	0.50	0.5000	0.0000
Silty Clay	1.2062	1.2024	0.1002	0.0038	118.7938	-0.1515	0.50	0.5010	-0.0010
Clay	0.9027	0.9012	0.0751	0.0015	119.0973	-0.1507	0.50	0.5007	-0.0007
			Averages:	0.0119		-0.1649			-0.0012

[1] Based on TOL = 0.001 ft

Generated externally using *HEC-HMS* and hard-coded in spreadsheet
Generated externally using *FLO-2D* and hard-coded in spreadsheet
Generated externally using *HEC-1* and hard-coded in spreadsheet

This test model simulates a uniform flow depth over time except for the initial two hours when flow is increased from 0 to 1,500 cfs. The uniform depth of flow was taken from the HYCHAN.OUT file and is the average of the maximum uniform flow depths, neglecting the initial run-up. The *FLO-2D 1D* channel infiltration volume of 2.81 ac-ft for the run was taken from the CHVOLUME.OUT and SUMMARY.OUT files and is reported in Table 4.7. The verification unit infiltration volume of 13.0638 inches from the *G&A Computation Program* was converted to total infiltration in ac-ft as follows:

$$V_i = \frac{L_c * W_c * \frac{I_m}{12}}{43{,}560}$$

= (950*117.35*13.0638/12)/43560 = 2.786 ac-ft

where:

V_i = Total computed infiltration volume in ac-ft

L_c = Length of channel = 95 grids*10 feet = 950 feet

W_c = Wetted perimeter of channel width = 11 grids*10 feet+2*D_a = 11*10+2*3.677 = 117.35 feet

D_a = Average uniform flow depth = 3.677 feet (computed from *FLO-2D* results)

I_m = Infiltration volume computed by the *G&A Computation Program* = 13.0638 inches

Scenario 2: Inflow Only Model - Infiltration by reach – Cross Sections - *LID*

This example is identical to Scenario 1A except that natural ground cross sections were used to define the rectangular channel. The results of this test were identical to Scenario 1A and are listed in Table 4.5.

Table 4.7 *1D* Channel *LID* Testing Results

Scenario	Infiltration Volume			
	***FLO-2D*, ac-ft**	***G&A* Computation Program**		**Difference, ac-ft (%)**
		inches	**ac-ft**	
(1)	**(2)**	**(3)**	**(4)**	**(5)**
1: Inflow only, infiltration by reach, prismatic section	2.81	13.0638	2.786	0.024 (0.86)
2: Inflow only, infiltration by reach, cross sections	2.81	13.0638	2.786	0.024 (0.86)

The minor difference of 0.86% between the *FLO-2D* infiltration volume and the independent check is acceptable for FCDMC purposes.

4.2.1.10 Rainfall-Runoff from Impervious Obstructions

Effects of Impervious Area on Infiltration

When the percentage of impervious area on a grid element is less than 100%, the remaining pervious area is used by *FLO-2D* to compute the volume of infiltration for each time step. This is difficult to check for a rainfall-runoff model using independent methods because *FLO-2D* is using the head term and computing infiltration based on flow depth, which changes for every time step. Therefore, a simple check was done using a modified version of the level pool model described in Section 4.2.1.6 with the RTIMP parameter for Grid 41 set to 50% (0.5). The results were then compared with RTIMP set to 0% (0.0) for Grid 41. The check was made using the final depth for the grid, subtracted from the initial depth of 10.0 feet, which is the starting level pool depth. The 50% *RTIMP* model should produce about one half of the decrease in depth that the 0% *RTIMP* model does. The results are listed in Table 4.8. The results match the expected reduction in ponded depth. The higher *XKSAT* value texture classes have up to a 2% higher reduction because the ponding depth is deeper for a longer period of time than the 0% *RTIMP* model. This allows for a higher pressure head, which infiltrates a little more than 50% of the 0% *RTIMP* model.

Table 4.8 Infiltration Volume Difference for 50% Impervious Area

	Final Depth, ft		**% Reduction in Volume**
	Grid Percent Impervious (*RTIMP*)		
Texture Class	**0%**	**50%**	**[10-(3)]/[10-(2)]*100**
(1)	**(2)**	**(3)**	**(4)**
Loamy Sand and Sand	7.66	8.79	52%
Sandy Loam	8.71	9.34	51%
Loam	9.00	9.49	51%
Silty Loam	9.17	9.58	51%
Silt	9.37	9.68	50%
Sandy Clay Loam	9.58	9.79	50%
Clay Loam	9.66	9.83	50%
Silty Clay Loam	9.63	9.81	50%
Sandy Clay	9.79	9.89	50%
Silty Clay	9.79	9.89	50%
Clay	9.87	9.93	50%

<u>Buildings Defined Using *ARF* Covering Multiple Grids</u>

Buildings can be defined in *FLO-2D* as described in Section 2.3.3.4. The *FCDMC* typically models rainfall-runoff from buildings using method 2 (partially or completely blocked grid, *ARF*≤1), which is applied using the *ARF* field on the *WRF* record. Separate models for method 1 were not prepared, but that approach also works correctly (refer to Section 4.2.3). Refer to Sections 2.4.6 and 3.3. Method 2 was tested using two (2) simple model scenarios. The models were created using the following common conditions:

1. 10-foot grid size with 1836 grids laid out in the rectangle. The surface was set to drain from north to south at a uniform slope of 0.5 percent.
2. Contiguous grids 958, 959, 1009, and 1010 were assigned *ARF* equal to one (1), which completely blocks each grid. The *WRF* fields were set to zero (0).
3. *IRAINBUILDING* set equal to one (1) in RAIN.DAT, which turns on rainfall-runoff from buildings.
4. Infiltration set to zero (0) in CONT.DAT, which turns infiltration off.
5. *IWRFS* set to one (1) in CONT.DAT, which turns on the blockage function.
6. *TOL* set to 0.01 feet.
7. *IRAINARF* set equal to one (1) in RAIN.DAT, which indicates that individual grid depth-area reduction values are assigned.
8. Rainfall set to occur only on grids 958, 959, 1009, and 1010 by setting *RAINARF* equal to one (1), and zero (0) for all other grids.
9. Rainfall set to 5-inches in 6-hours using *DDM Hydrology* storm pattern 1.

The two model scenarios are described as follows:

<u>Rainfall on Building Scenario 1</u>. This test was designed to verify that building grids with *ARF* set equal to one (1) share flow within the building and that all runoff volume in excess of *TOL* drains from the building onto the adjacent floodplain grids. This was accomplished by defining levees around the outside of the four building grids set to elevation 1000, approximately 500 feet in height. Refer to Figure 4.2. The levees are represented by red lines, the *ARF* grids are shaded grey, and the side blockages resulting from the *ARF* setting of one (1) are shown as orange lines. The building grid elevations were set to drain to the southeast corner grid (1009). The south side levee for grid 1009 was left open. Grid 1008 immediately downstream of grid 1009 was set to be a 3 foot deep sump using elevation 499.0. Levees were assigned to enclose grid 1008 except for the north side. Rainfall on the four building grids drains into sump grid 1008.

Scenario 1 Test Measure: The final depth in grid 1008 should be equal to the sum of the rainfall from the four building grids minus the depth of *TOL* times four.

The results of Scenario 1 are shown on Figure 4.3. The final depth in grid 1008 is 1.63 feet. The check is:

$$D_f = \left(\left(\frac{R_d}{12}\right) - TOL\right) * 4$$

$$D_f = \left(\left(\frac{5}{12}\right) - 0.01\right) * 4 = 1.63\ feet$$

where:

D_f = final depth in grid 1008, feet
R_d = Total rainfall depth = 5-inches
TOL = 0.01 feet

Based on this test, rainfall-runoff between building *ARF* grids and drainage off the building functions as expected.

Rainfall on Building Scenario 2. This test was designed to verify that flow external to the building will not be allowed to enter the building. Scenario 1 was modified to remove the levees around building grids 958, 959, 1009 and 1010. A 50 cfs uniform flow hydrograph was introduced at both grids 969 and 1020. Refer to Figure 4.4 for a plan view of Scenario 2. The green cross hatching covers the two inflow grids.

Scenario 2 Test Measures: a. Runoff from the building grids should almost all drain to the sump at grid 1008. b. The external runoff from upstream should drain around the building and not enter the building or grid 1008.

The results for Scenario 2 are shown on Figure 4.5. Note that the maximum and final depths within the building and on grid 1008 are the same as for Scenario 1. Also note that external flow does not enter the building as attested to by the velocity vector arrows and the flow depths. Based on this test, rainfall-runoff between building *ARF* grids and drainage off the building in combination with blockage of external flow, functions as expected.

Figure 4.2 Building Runoff Scenario 1 Plan

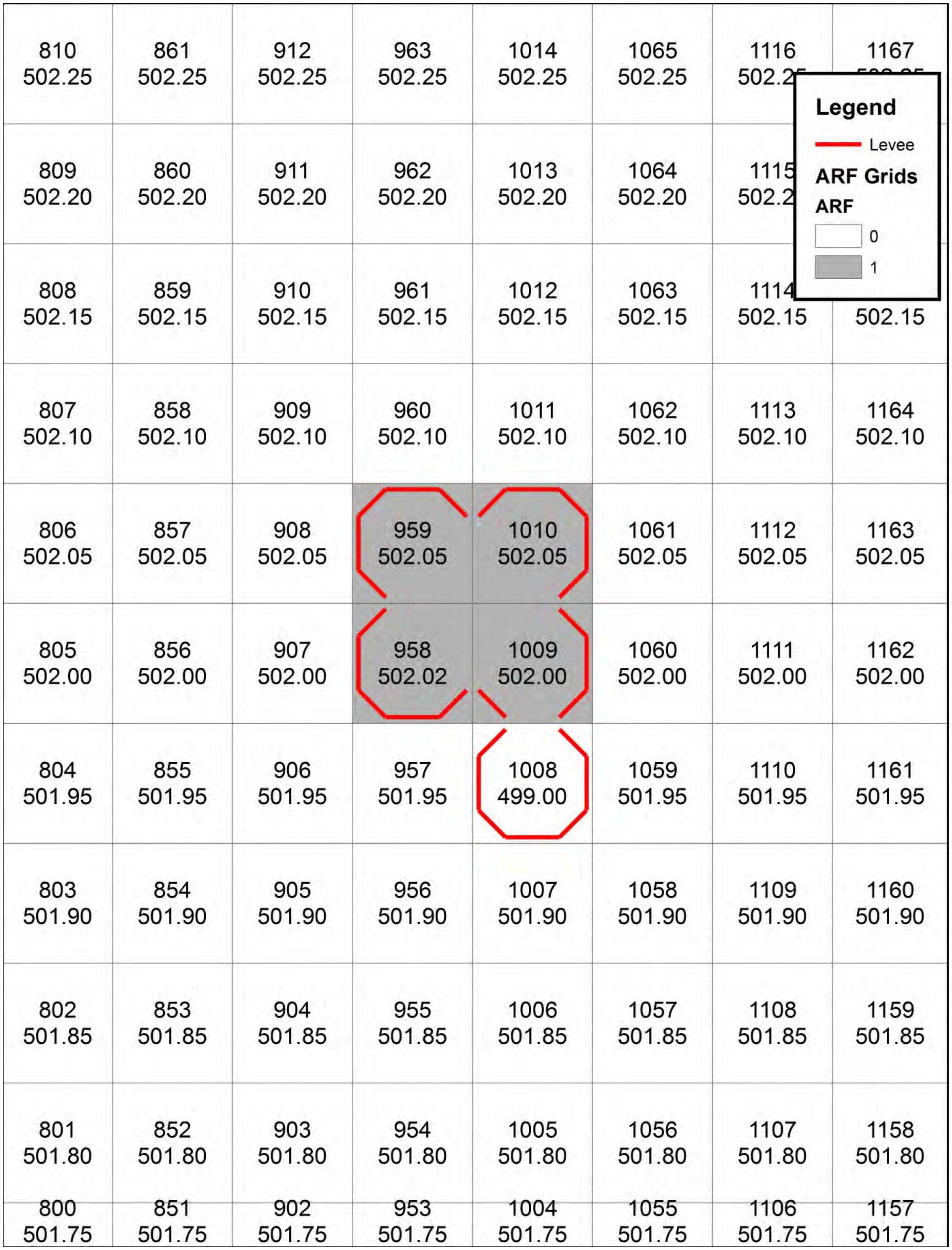


Figure 4.3 Building Runoff Scenario 1 Final Depth Results

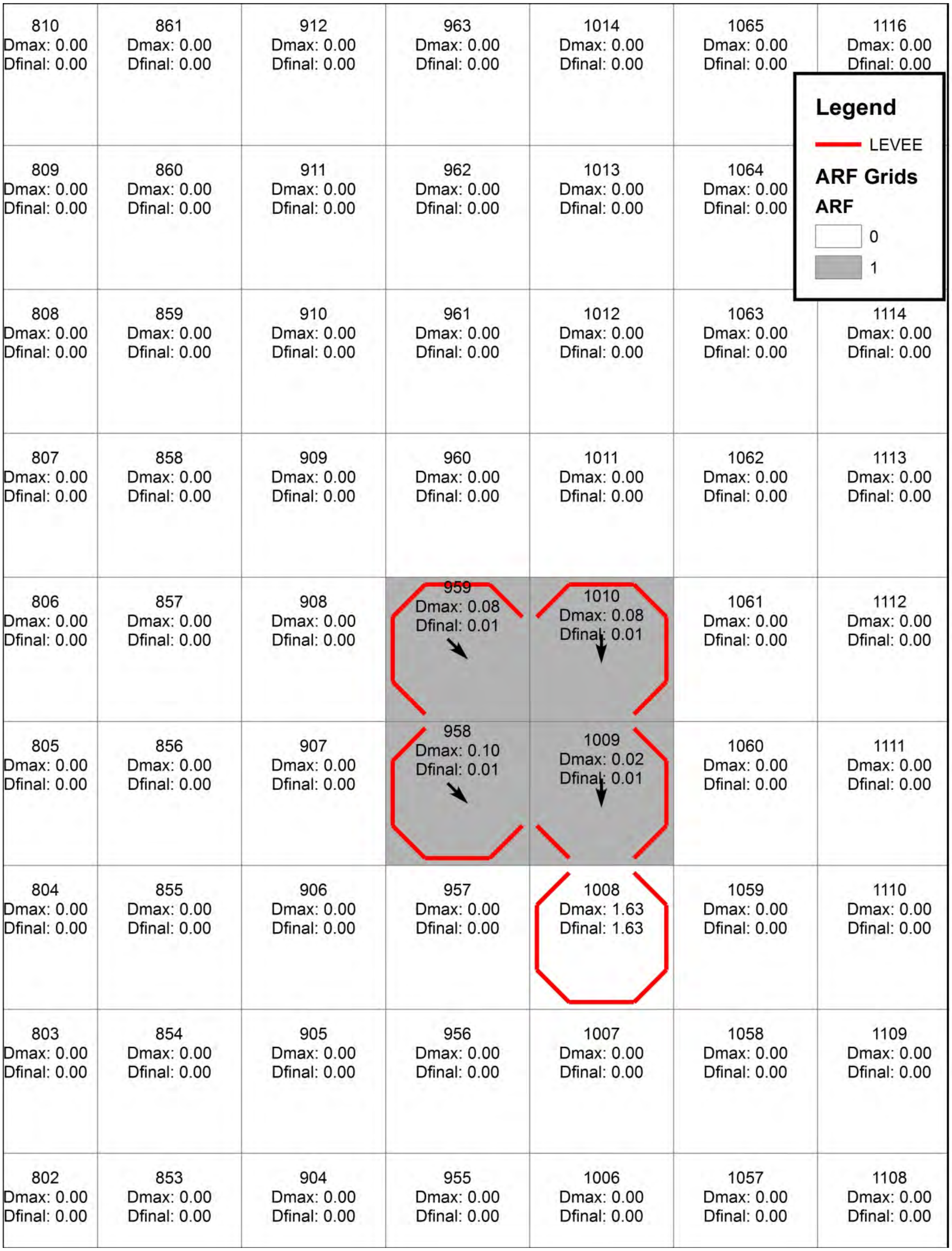


Figure 4.4 Building Runoff Scenario 2 Plan

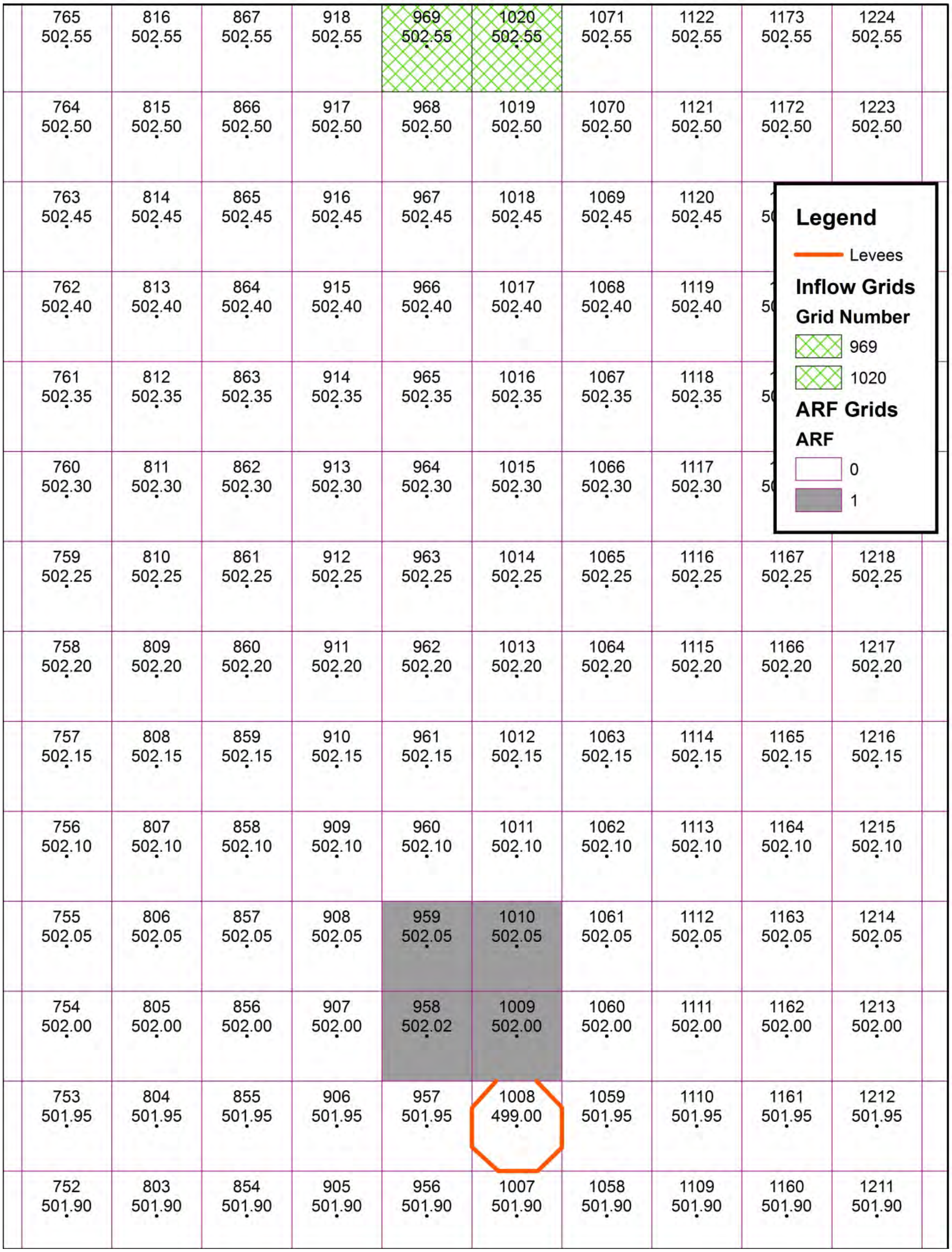
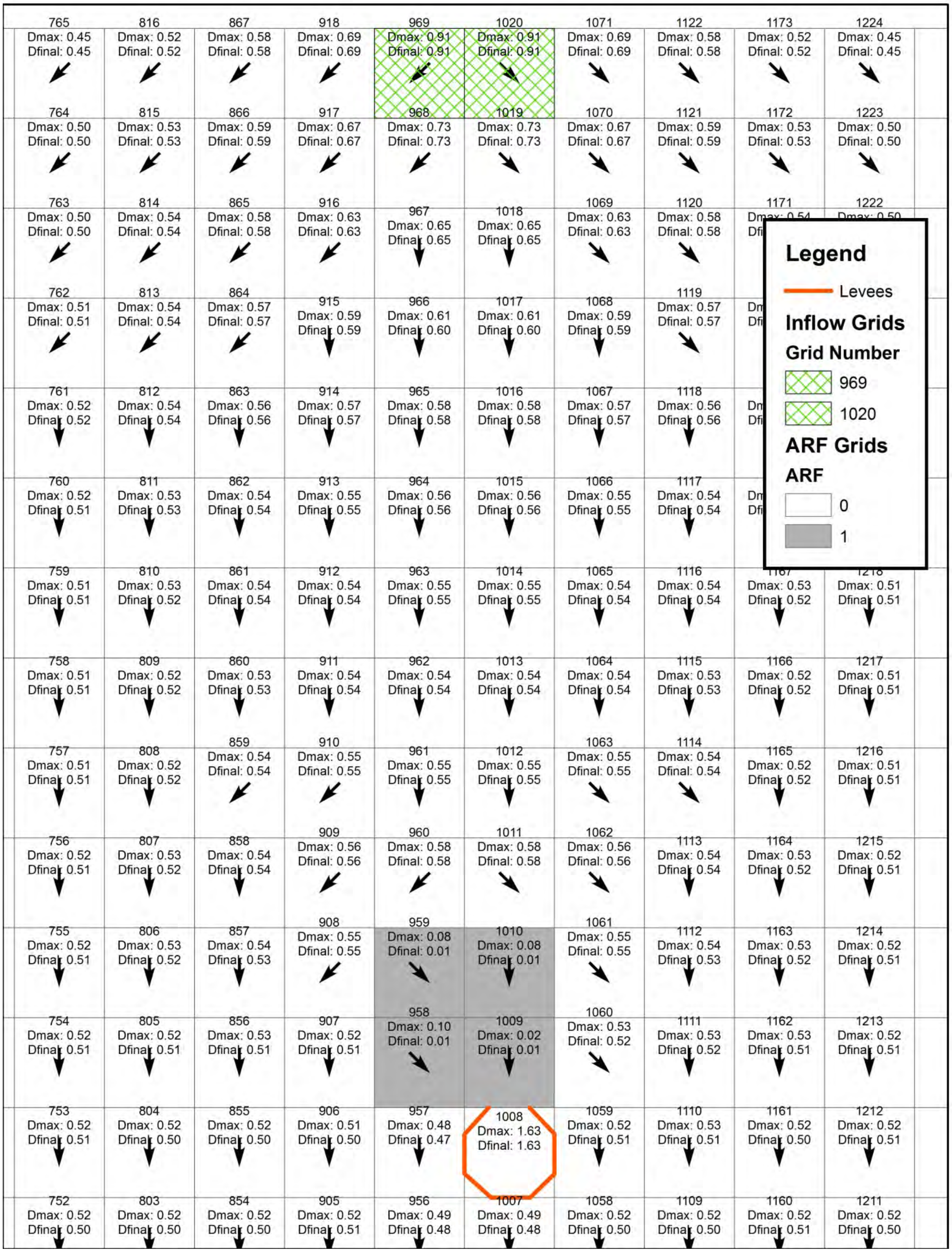


Figure 4.5 Building Runoff Scenario 2 Maximum Depth and Final Depth Results



4.2.2 Hydraulic Calculations Comparison with *HEC-RAS*

4.2.2.1 General

The purpose of this section is to verify that hydraulic calculations between grid elements performed by *FLO-2D* are done appropriately by comparing the results with the *USACE HEC-RAS* computer program. Verification models were prepared for the following scenarios:

1. Flume Model – Grid Based: A flume type channel simulated with floodplain grids.
2. Flume Model – *1D* Channel (isolated): A flume simulated using the *FLO-2D 1D* channel option. Inflow and outflow is limited to the channel only.
3. Flume Model – *1D* Channel (upstream grids): A flume simulated using the *FLO-2D 1D* channel option with inflow transitioning from floodplain grids into the *1D* channel.
4. Flume Model – *1D* Channel (downstream grids): A flume simulated using the *FLO-2D 1D* channel option with outflow transitioning from the *1D* channel onto downstream floodplain grids.
5. Rainbow Wash Natural Channel. *1D* channel simulation of a 5.5 mile long natural channel in south central Maricopa County.

4.2.2.2 Scenario 1: Flume Model – Grid Based

The purpose of this test is to verify that hydraulic calculations between grid elements are performed appropriately. This test consists of a rectangular flume with the following characteristics:

1. Base Width: 94.142 feet for *HEC-RAS*, 100.00 feet for *FLO-2D* (see discussion below),
2. Depth: 10 feet,
3. Slope: uniform at 0.2%,
4. *n*-value: 0.040, and
5. Length: 950 feet.

The flow regime is sub-critical. A steady state *HEC-RAS* model with the above characteristics was built using cross sections at 10 foot intervals and uniform peak discharges of 500, 1,500, and 5,000 cfs. A *3D* view of the *HEC-RAS* flume for 1,500 cfs is shown on Figure 4.6. The results of this model supply the benchmark for testing *FLO-2D*. A steady state *HEC-RAS* model was used for this case to show that an unsteady *FLO-2D* model can replicate steady state *HEC-RAS* results at the common peak discharge. This is important since steady state *HEC-RAS* is used as the basis for most *FEMA* floodplain delineation studies. Refer to Section 4.2.5 for numerous test cases that include comparing *FLO-2D* results with unsteady *HEC-RAS* results.

A *FLO-2D* model was built using 1,164 uniform square 10-foot grid elements. The flume was modeled by insetting a rectangular channel simulated with 10 grid elements forming the base, 10 feet below the grade of the adjacent elements as shown on Figure 4.7.

Figure 4.6 Flume Model Using *HEC-RAS*

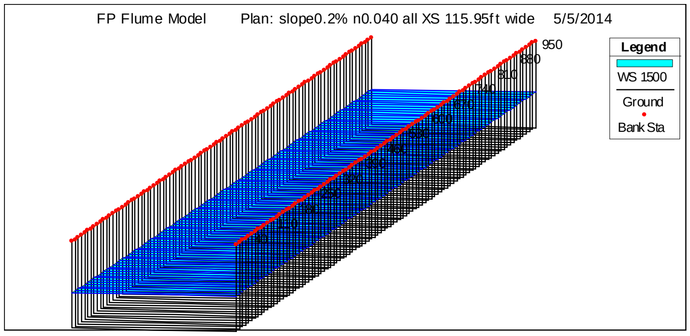
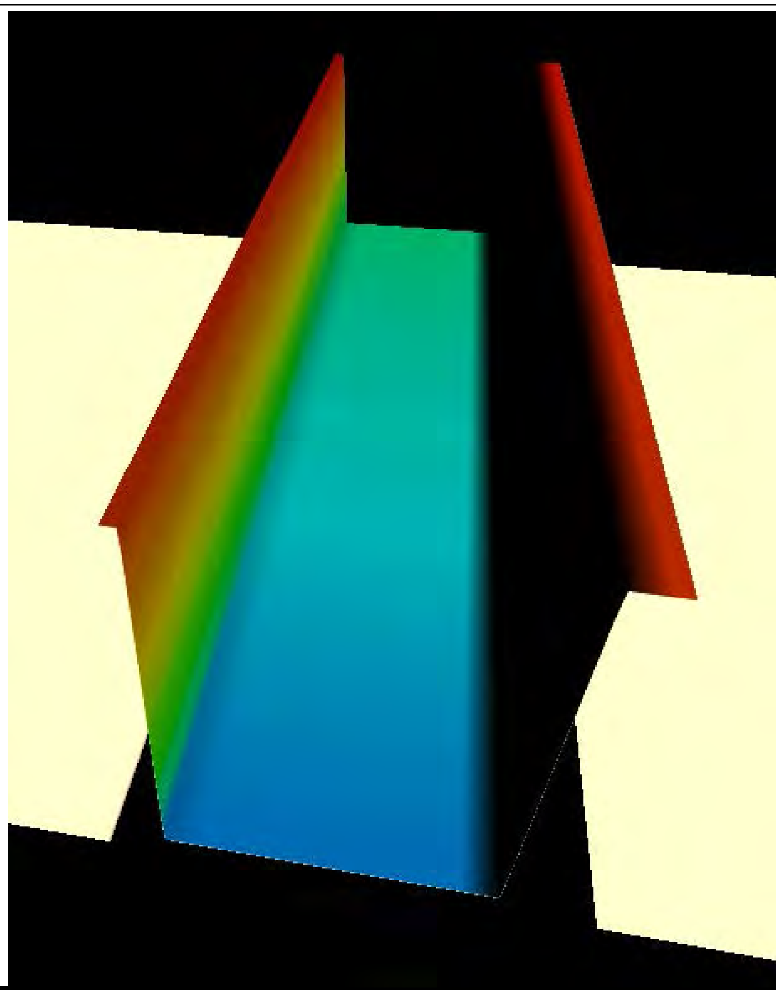


Figure 4.7 *FLO-2D* Flume Model Surface Using Floodplain Grids



The slope of the ground surface and the channel was set at 0.2% and Manning's *n* was set to 0.040 for all grid elements. The *FLO-2D* flume has a base width of 100 feet. The *HEC-RAS* model was created with a base width of 94.142 feet. This difference is intentional in order to make a fair comparison between the two models. The following is a description of why this is necessary:

> When a set of grid elements form a confined flow area, *FLO-2D* cannot exchange flow with an adjacent grid that is higher in elevation than the flow depth on the grid. For this case, the flume is aligned in a due north to south direction so that the grid sides are parallel to the flume. Therefore, the grids that are immediately adjacent to the flume sides cannot exchange flow with the bank grids, which are 10 feet high.
>
> For example, consider the row of grids shown in Figure 4.8 defined by bank grids 55 and 1122. The bank grids are shaded brown. Flow cannot be exchanged from grid 152 to grid 55 in directions 7, 4, and 8 and flow cannot be exchanged from grid 1025 to grid 1122 in direction 5, 2 and 6. For this example, the flow direction is forced from north to south because the flow is confined. Therefore, referring to Figure 4.8, the conveyance portion of grid 152 associated with direction 7 is not effective. The width of the ineffective area (FLO-2D Software, Inc., 2012a) (FLO-2D Software, Inc., 2014a) is (1-0.4142)/2 times the grid width of 10 feet, or 2.929 feet. Refer to Figure 4.9 for a diagram of the *FLO-2D* grid 8-sided representation. The same is true for grid 1025, so the total effective conveyance width used by *FLO-2D* is 100 feet – 2*2.929 = 94.142 feet. This is why the *HEC-RAS* model flume base width was set to 94.142 feet.

In order to be able to directly compare the *FLO-2D* results with *HEC-RAS*, the *FLO-2D* functions that adjust *n*-value must be turned off. The limiting *Froude number* was set high in order to avoid any *n*-value adjustments. Also, the depth-variable *n*-value routine was turned off (*AMANN* in CONT.DAT). Inflow to the *FLO-2D* model was spread out evenly over 10 grids by dividing the total inflow hydrograph ordinates by 10 and applying the resulting hydrograph to each of the 10 grid elements. The *HEC-RAS* model was run in subcritical mode. Important *FLO-2D* input control settings used were:

1. Limiting *Froude number* set to 2.0,
2. Shallow *n* set to 0.00,
3. AMANN set to -99,
4. Floodplain COURANTFP set to 0.10, and
5. *TOL* set to 0.1.

The results from the *FLO-2D* and *HEC-RAS* models shown in Table 4.9. The average depth and velocity results for selected *HEC-RAS* cross sections and the corresponding *FLO-2D* grids are comparable but slightly different between the two models.

Figure 4.8 *FLO-2D* Flume Model Grid Elevations

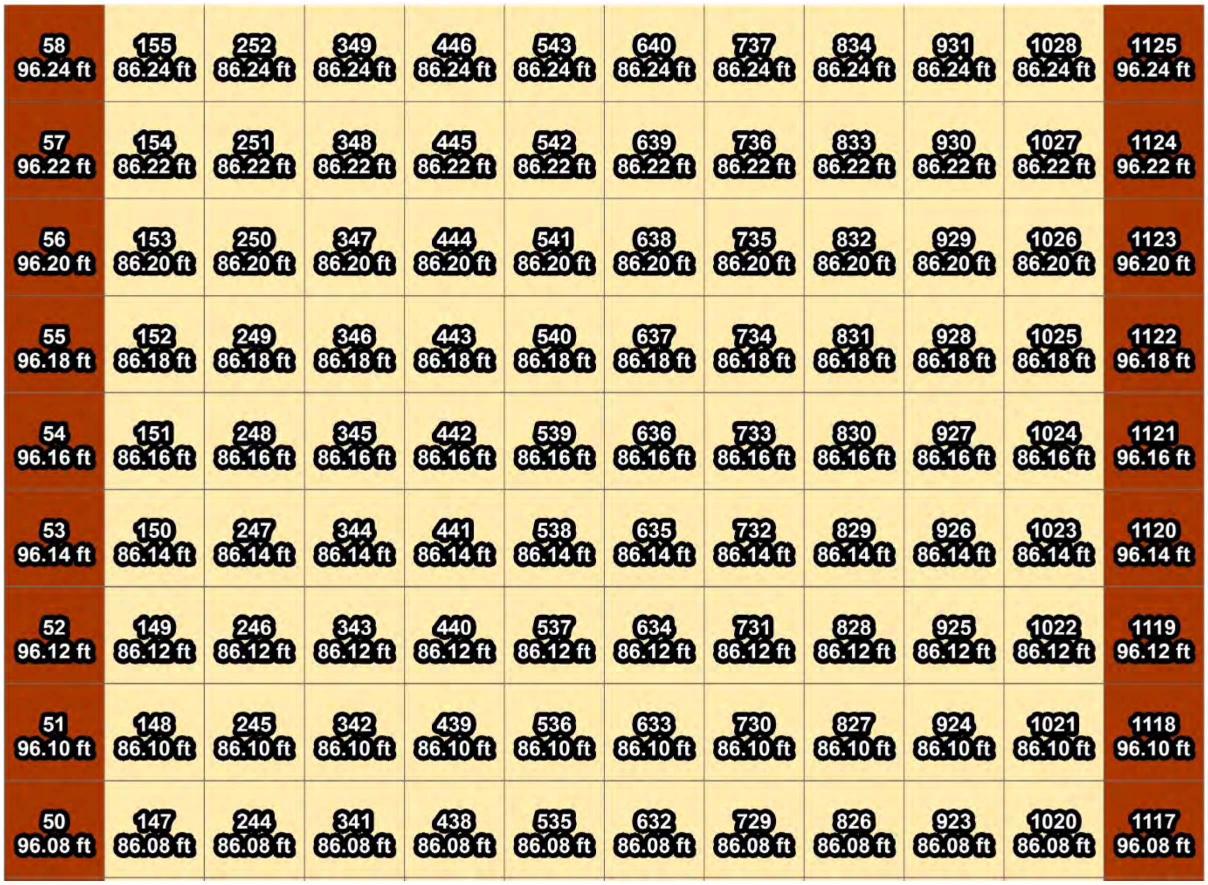
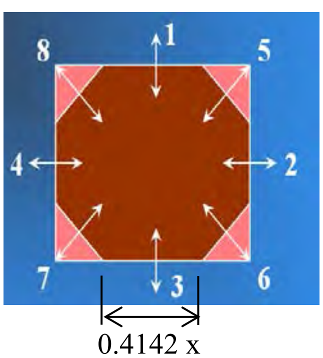


Figure 4.9 *FLO-2D* Grid Diagram



***FLO-2D* Grid Possible Flow Directions for Grid Size x**

FLO-2D is an unconfined flow model, so the wetted perimeter is assumed to be only the base width. This results in a slightly higher velocity and slightly lower flow depth when compared with *HEC-RAS*. Also, slight differences are to be expected because the computation methods between a *1D* model and a grid-based *2D* model are dissimilar.

$$V = \frac{1.486\left(\frac{A}{P}\right)^{0.67} S^{0.52}}{n}$$

For *FLO-2D* at 1,500 cfs:

$$V = \frac{1.486\left(\frac{94.142 * 3.89}{94.142}\right)^{0.67} 0.002^{0.52}}{0.040} = 4.11\ ft/s$$

For *HEC-RAS* at 1,500 cfs:

$$V = \frac{1.486\left(\frac{94.142 * 4.01}{94.142 + 2 * 4.01}\right)^{0.67} 0.002^{0.52}}{0.040} = 3.97\ ft/s$$

The above calculations demonstrate that the small differences in results between FLO-2D and HEC-RAS for this scenario are primarily due to the difference in wetted perimeter.

The conclusion is that *FLO-2D* floodplain grid element method performs hydraulic calculations in an acceptable manner. If a prismatic channel is to be modeled using only inset *FLO-2D* grid elements, the lack of conveyance of a portion of each bank grid element should be accounted for by the modeler by assigning and appropriate *ARF* value. If the wetted perimeter including only the base width is an issue for a modeled prismatic channel, then the *1D* channel option should be applied.

Table 4.9 Comparison of *FLO-2D* Grid Based Channel with *HEC-RAS*

***HEC-RAS* River Station**	***FLO*-2D Grid Number**	**Peak Q = 500 cfs**				**Peak Q = 1500 cfs**				**Peak Q = 5000 cfs**			
		Depth		**Velocity**		**Depth**		**Velocity**		**Depth**		**Velocity**	
		RAS	**2D**	**RAS**	**2D**	**RAS**	**2D**	**RAS**	**2D**	**RAS**	**2D**	**RAS**	**2D**
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**	**(11)**	**(12)**	**(13)**	**(14)**
400	542	2.04	2.01	2.60	2.65	4.01	3.89	3.97	4.12	8.55	8.00	6.21	6.74
900	542	2.04	2.01	2.60	2.65	4.01	3.89	3.97	4.12	8.55	8.00	6.21	6.74
1200	542	2.04	2.01	2.60	2.65	4.01	3.89	3.97	4.12	8.55	8.00	6.21	6.74

4.2.2.3 Scenario 2: Flume Model – *1D* Channel (isolated)

The purpose of this test is to verify that hydraulic calculations for the *FLO-2D 1D* channel option are performed appropriately. The *HEC-RAS* model described in Section 4.2.2.2 was used as the benchmark model for this test. Refer to Figure 4.10 for the *FLO-2D 1D* channel layout. The *HEC-RAS* cross sections are spaced 10 feet apart and line up with the center of each *FLO-2D* grid element and the base width was changed to 100 feet. The *FLO-2D* channel COURANTC was set to 0.10, and *TOL* was set to 0.001. For this case, all flow is contained within the *FLO-2D 1D* channel using channel inflow and outflow grids. Flow does not transition from the floodplain to the channel or from the channel to the floodplain. The flow regime is sub-critical.

The results from the *FLO-2D* and *HEC-RAS* models are compared in Table 4.10 and Table 4.11. The average depth and velocity results for selected *HEC-RAS* cross sections and the corresponding *FLO-2D* grids are shown in Table 4.10. There are virtually no differences in flow depths between *FLO-2D* and *HEC-RAS* for all three peak discharges. As an additional check, the average cross sectional area and velocity for all *HEC-RAS* cross sections and *FLO-2D* cross sections were computed and are listed in Table 4.11. These values were then used to compute the discharges shown in column 5 for comparison with the input discharges listed in column 2. The results are nearly identical.

The conclusion is that the *FLO-2D 1D* channel method performs hydraulic calculations in an acceptable manner and is suitable for use on *FCDMC* projects.

4.2.2.4 Scenario 3: Flume Model – *1D* Channel Transition to Downstream FP Grids

The purpose of this test is to verify that hydraulic calculations for the *FLO-2D 1D* channel transition to floodplain grids are performed appropriately. Figure 4.11 shows the plan view of the test model and Figure 4.12 shows the profile view.

The *FLO-2D* channel portion of the model has the following parameters:

1. Base Width: 49 feet
2. Depth: 10 feet
3. Slope: uniform at 0.2%
4. *n*-value: 0.040
5. Length: 910 feet

Figure 4.10 *FLO-2D* 1D Flume Model Grid Elevations

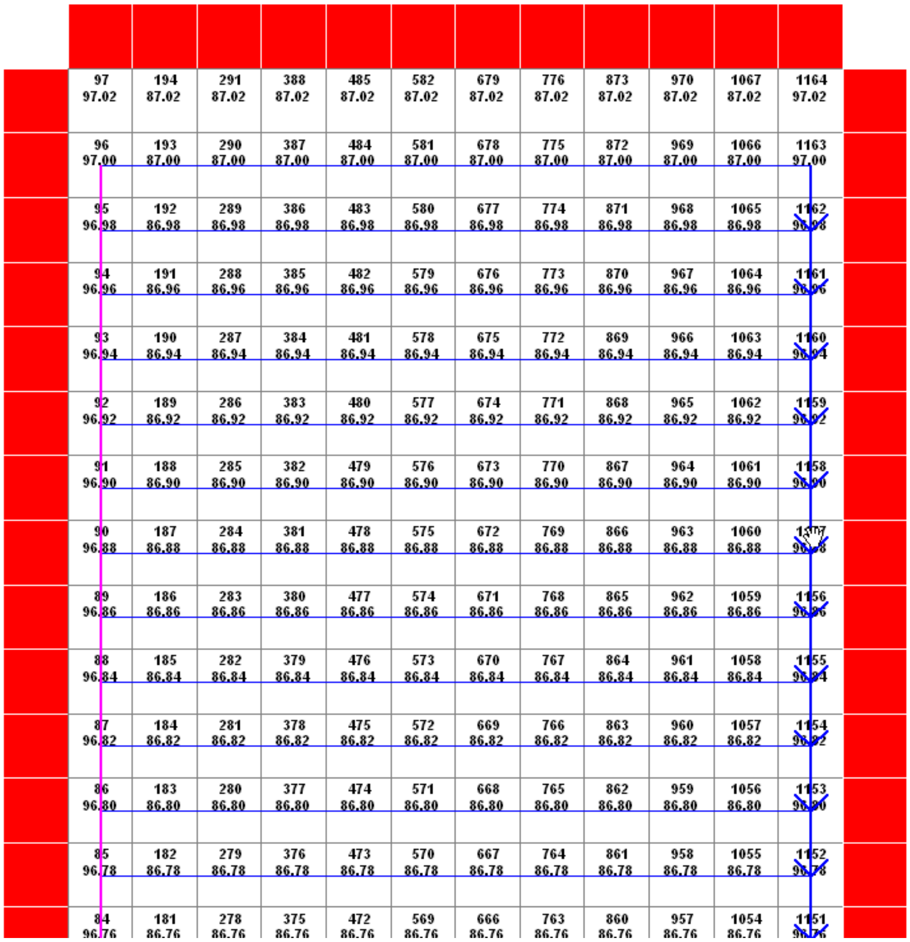


Table 4.10 Comparison of *FLO-2D 1D* Channel with *HEC-RAS*

HEC-RAS River Station	FLO-2D Grid Number	Peak Q = 500 cfs				Peak Q = 1500 cfs				Peak Q = 5000 cfs			
		Depth		Velocity		Depth		Velocity		Depth		Velocity	
		RAS	2D	RAS	2D	RAS	2D	RAS	2D	RAS	2D	RAS	2D
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
400	1108	1.97	1.97	2.54	2.53	3.86	3.86	3.89	3.89	8.18	8.19	6.10	6.11
600	1128	1.97	1.97	2.54	2.53	3.86	3.86	3.89	3.89	8.18	8.19	6.10	6.11
900	1158	1.97	1.97	2.54	2.53	3.86	3.86	3.89	3.89	8.19	8.19	6.10	6.11

Table 4.11 Comparison of Input Discharge to Calculated Discharge for *FLO-2D 1D* Channel

Model	Input Discharge cfs	Average Area sf	Average Velocity fps	Calculated Discharge (3) * (4)
(1)	(2)	(3)	(4)	(5)
HEC-RAS	500	197.00	2.54	500
FLO-2D	500	196.67	2.54	500
HEC-RAS	1500	386.00	3.89	1502
FLO-2D	1500	385.62	3.89	1500
HEC-RAS	5000	818.19	6.10	4991
FLO-2D	5000	818.94	6.11	5004

The *FLO-2D* channel transition section of the model has the following parameters:

1. Base Width: Transitions from 49 feet to 119 feet
2. Depth: Drops abruptly from 10 feet to 0.05 feet
3. Slope: uniform at 0.2%
4. *n*-value: 0.040
5. Length:40 feet

The *FLO-2D* floodplain flume section of the model starts at the beginning of the channel transition and has the following parameters:

1. Base Width: 160 feet
2. Depth: 10 feet
3. Slope: uniform at 0.2%
4. *n*-value: 0.040
5. Length: 1,080 feet

The *FLO-2D* was compared to a *HEC-RAS* model with the following parameters:

1. Base Width: 49 feet for 910 feet and 151.62 feet for 1,080 feet
2. Depth: 10 feet
3. Slope: uniform at 0.2 %
4. *n*-value: 0.040
5. Total Length: 1,990 feet

Figure 4.11 *FLO-2D 1D* Channel Transition to FP Grids Plan View

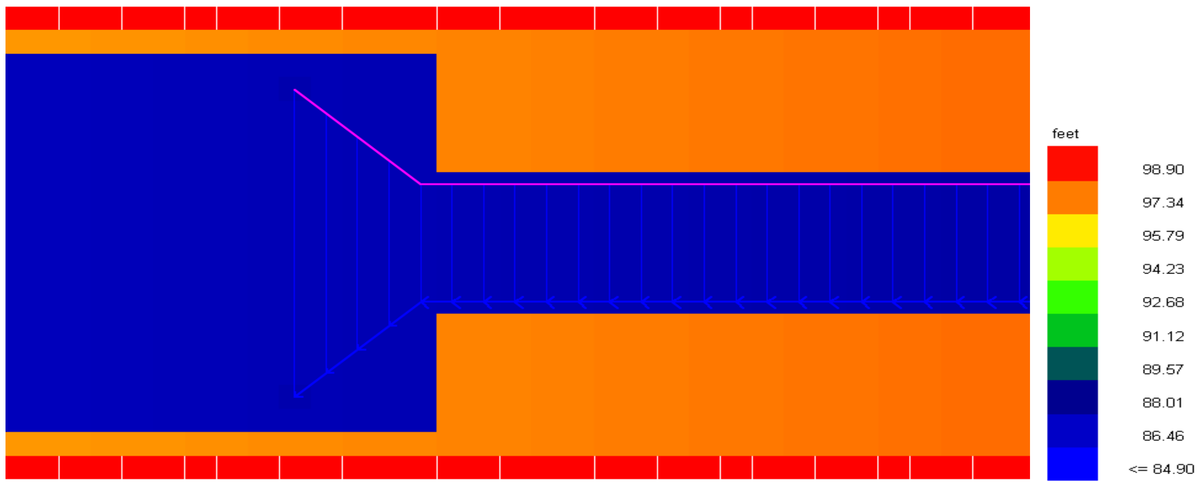
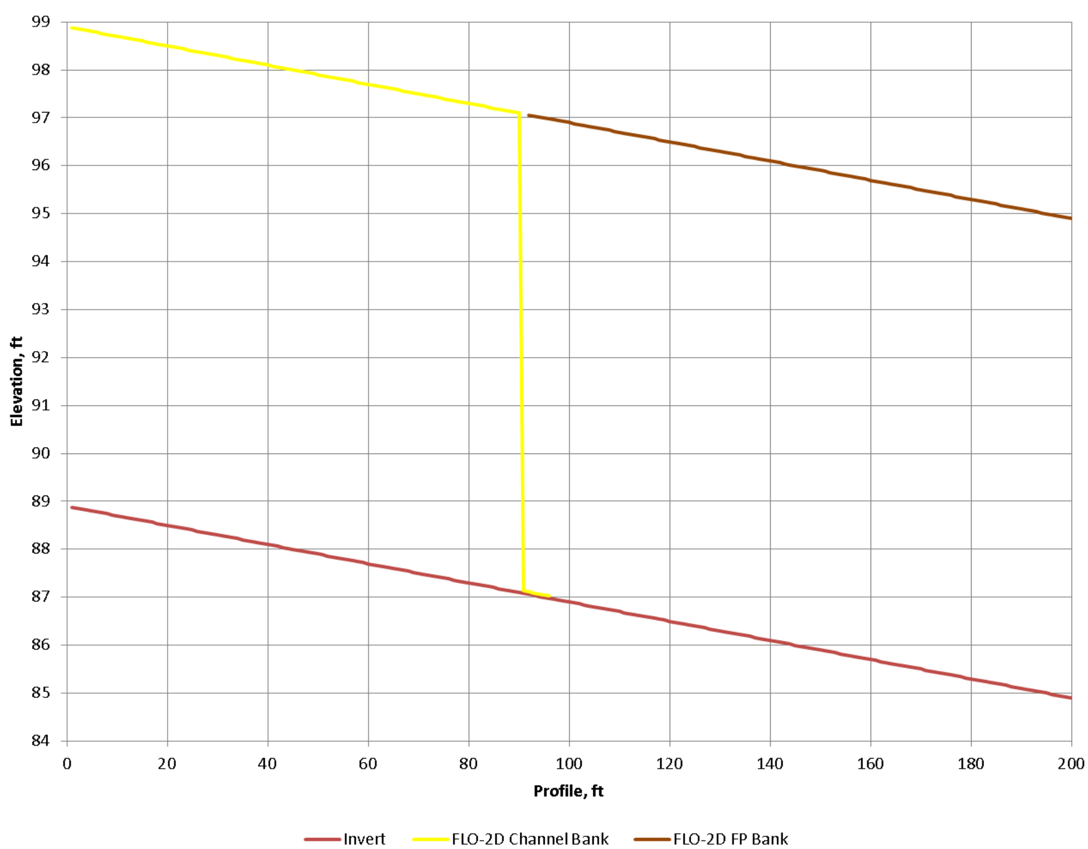
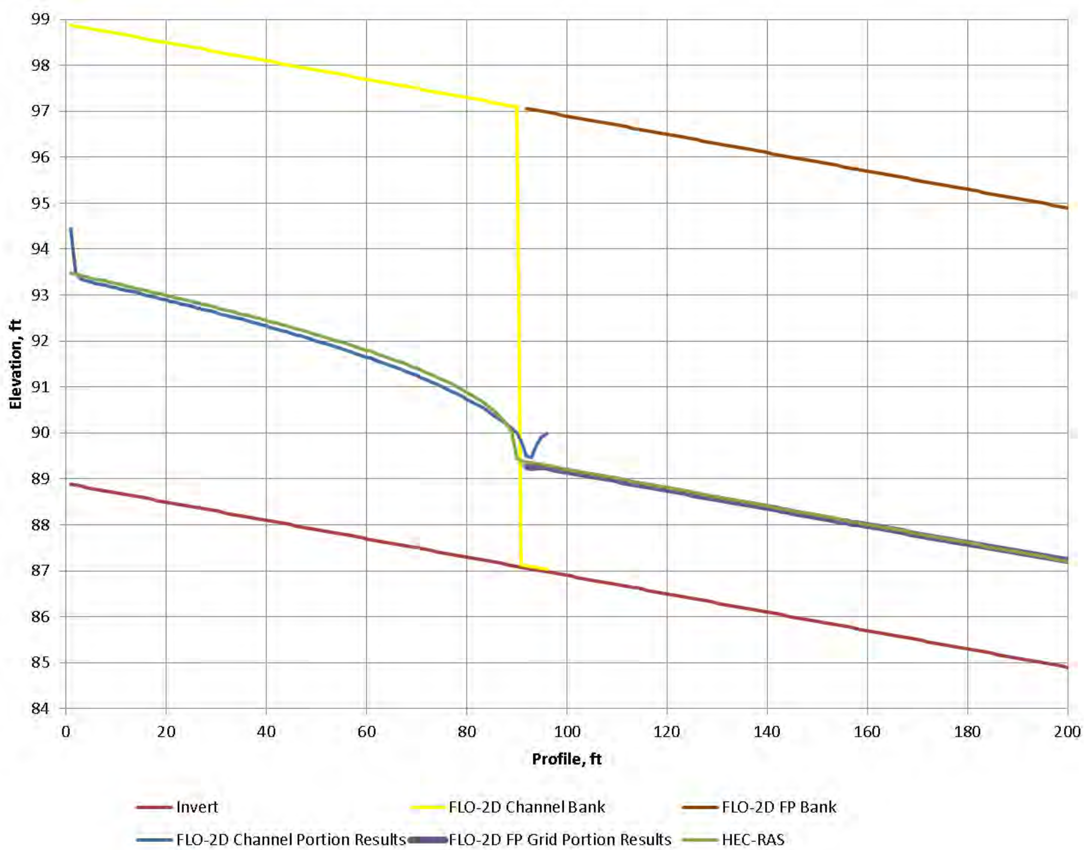


Figure 4.12 *FLO-2D 1D* Channel Transition to FP Grids Profile View



A comparison of the *FLO-2D* and the *HEC-RAS* model results is shown on Figure 4.13. The resultant *FLO-2D* water surface profile compares reasonably with the *HEC-RAS* water surface profile. This approach for modeling a *1D* transition to floodplain grids transitions as expected.

Figure 4.13 *FLO-2D 1D* Channel Transition to FP Grids Profile View



4.2.2.5 Scenario 4: Flume Model –Upstream Floodplain Grids Transition to *1D* Channel

The *FLO-2D* model does not have a specific method for accomplishing this transition. Flow from upstream floodplain grids cannot transition directly into the first *1D* channel cross section. Instead, flow backs up and flows around the first cross sections and enters the channel by flowing in over the channel banks. The modeler must be very careful when setting up floodplain grid to *1D* channel transitions.

4.2.2.6 Scenario 5: Rainbow Wash Natural Channel

<u>Purpose and Description</u>

The purpose of this verification example is to validate the *FLO-2D 1D* channel component using a natural channel by comparing unsteady *FLO-2D* model results with an unsteady *HEC-RAS* benchmark model.

The Rainbow Wash in Maricopa County was selected for this example. The location of the Rainbow Wash watershed is shown on Figure 4.14. The watershed for the study reach is shown on the Vicinity Map (Figure 4.15). The Rainbow Wash study reach is shown using an orange line on Figure 4.15. A new *HEC-RAS* base dataset was built in *HEC-GeoRAS* using available 2-foot topographic mapping. The *HEC-GeoRA*S data was imported into an unsteady *HEC-RAS* model as the base for this example. A total of 634 cross sections were defined using an average 50 foot spacing equalling the *FLO-2D* grid size of 50 feet. The cross sections only include the main channel and both left and right bank points. The overbank floodplain was not included in the model. A composite manning's *n* value of 0.035 was used in both *HEC-RAS* and *FLO-2D* for all cross sections.

Figure 4.14 Rainbow Wash Watershed LocationMap

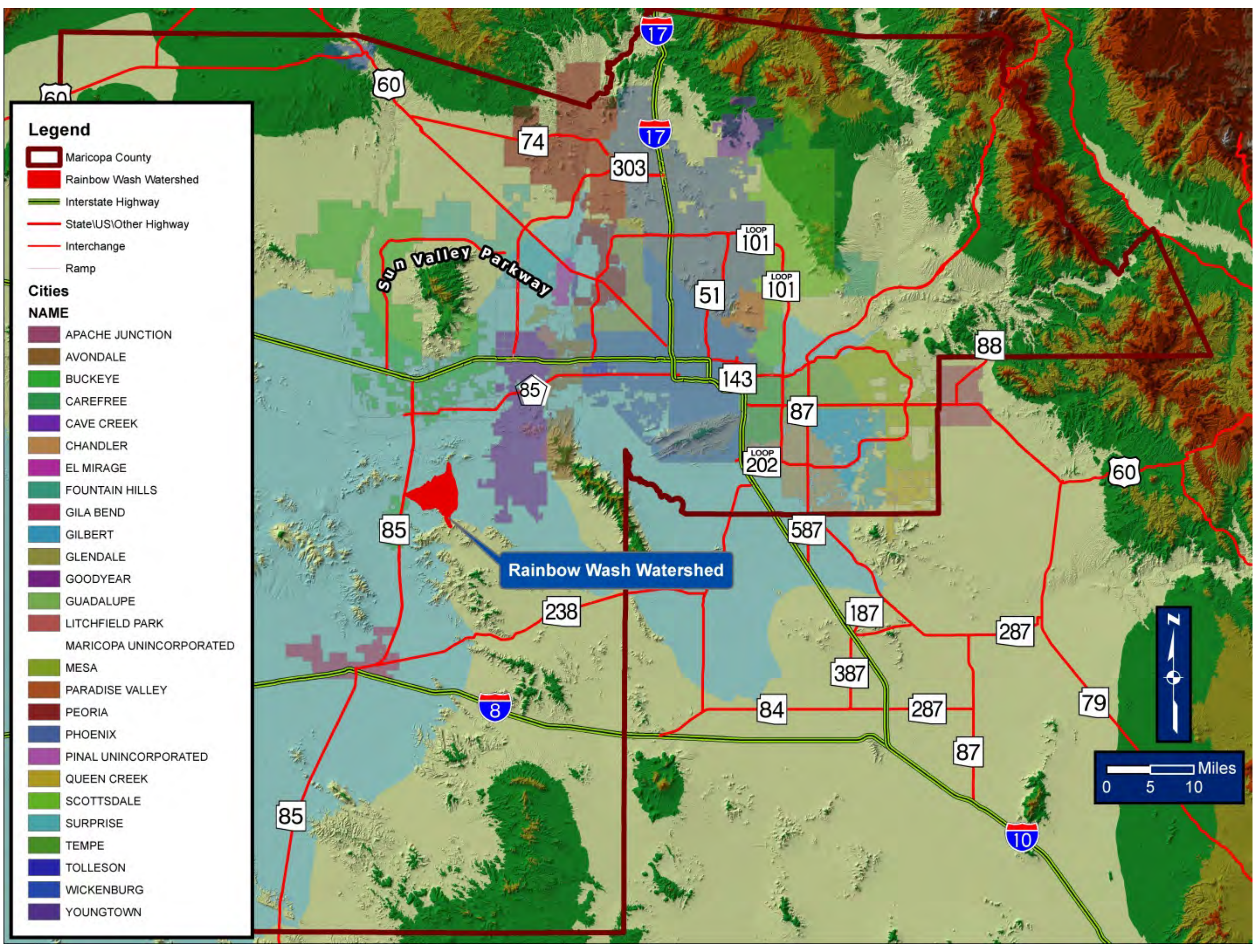
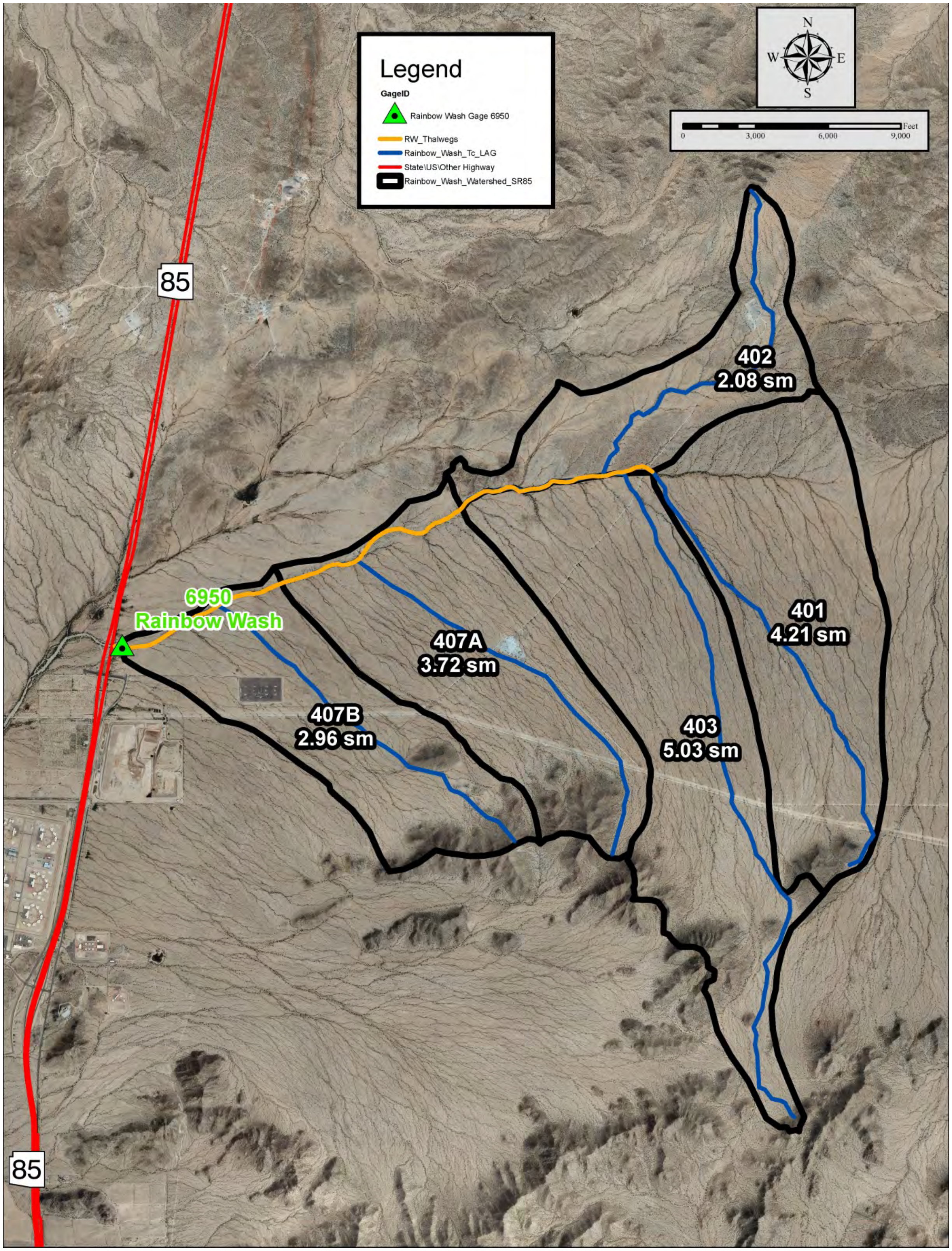


Figure 4.15 Rainbow Wash Vicinity Map



The model terrain data is shown on Figure 4.16. The reach modeled is approximately 5.5 miles in length. An example of the cross section layouts in relation to the *FLO-2D 1D* channel left bank grid elements is shown on Figure 4.17. After importing the data from *HEC-GeoRAS,* the cross sections were adjusted in *HEC-RAS* to define the left and right bank locations more accurately. Once the bank points were adjusted, the *HEC-RAS* cross section data was used to create the cross section data file (XSEC.DAT) for the *FLO-2D* model. Representative cross sections are shown on Figure 4.18. Note that the Rainbow Wash channel is generally trapezoidal in shape, fairly uniform, and increasing in width from upstream to downstream. It has some sinuosity, but it is not pronounced. The bed is typically sand with caliche underlying it at various depths. The bed is typically free of vegetation and the banks are usually heavily vegetated. A photograph of the channel in the vicinity of *HEC-RAS* Station 105.65 taken on July 6, 2011, is shown on Figure 4.19.

Figure 4.16 Rainbow Wash Topographic Surface

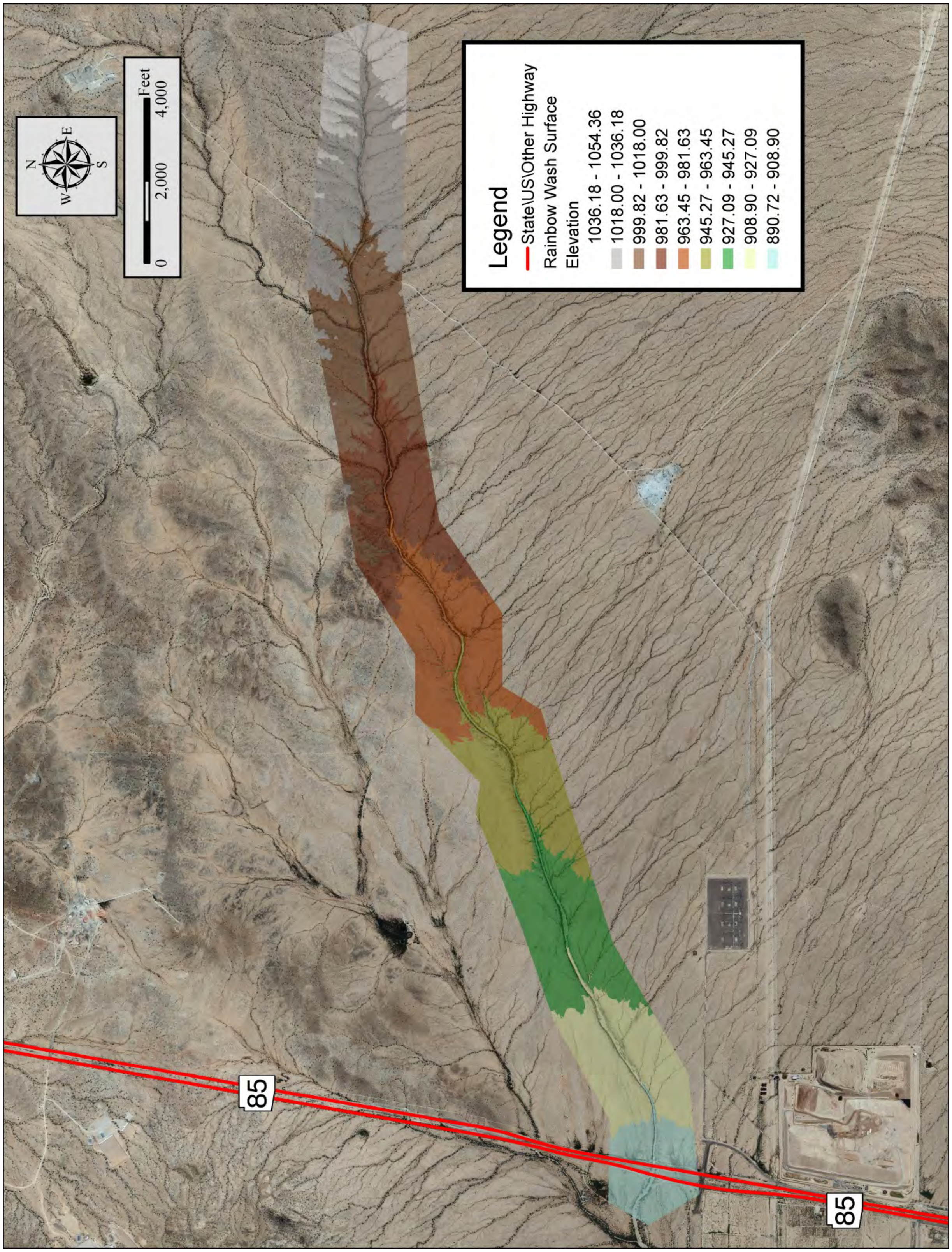


Figure 4.17 Rainbow Wash HEC-RAS Cross Section Locations Example

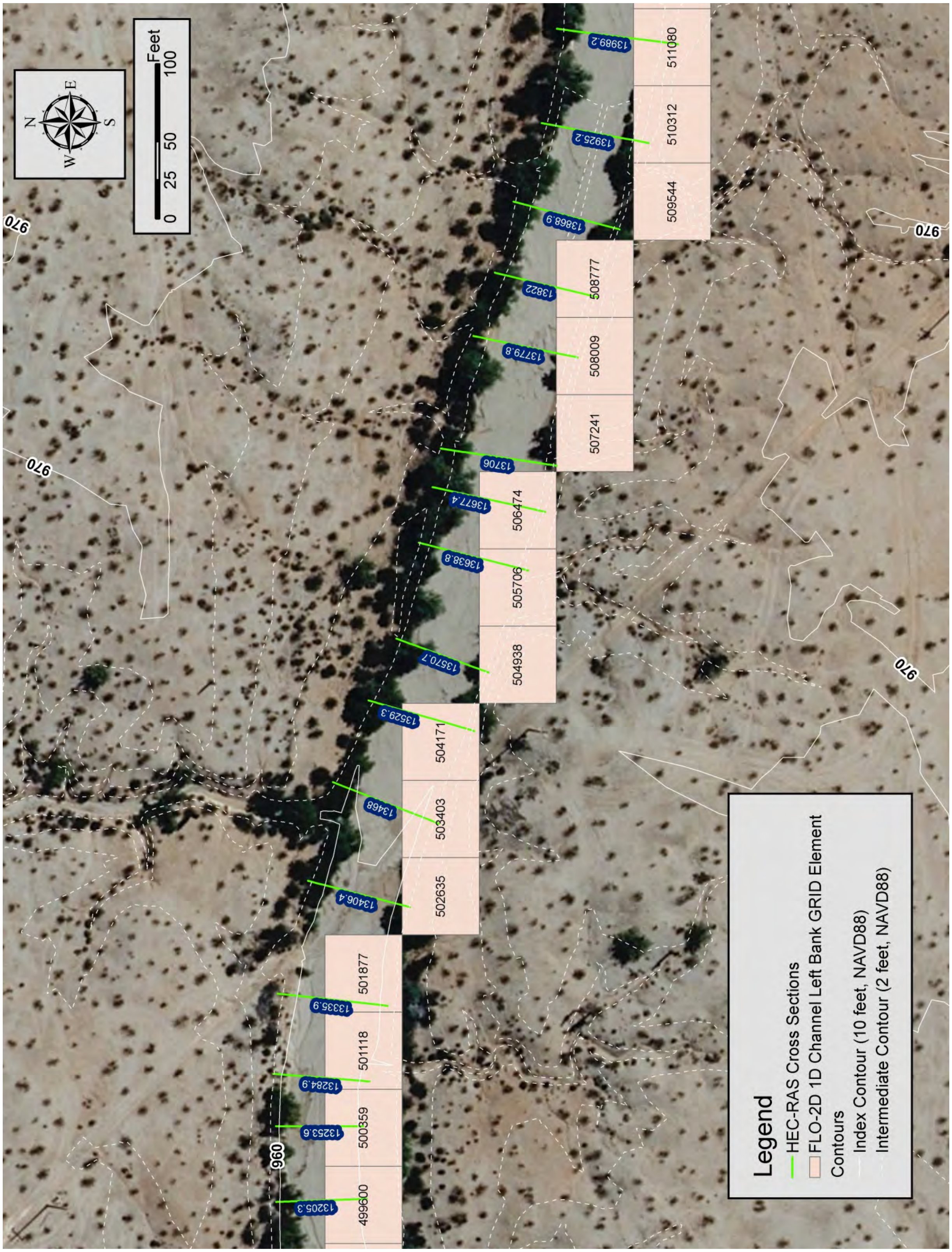
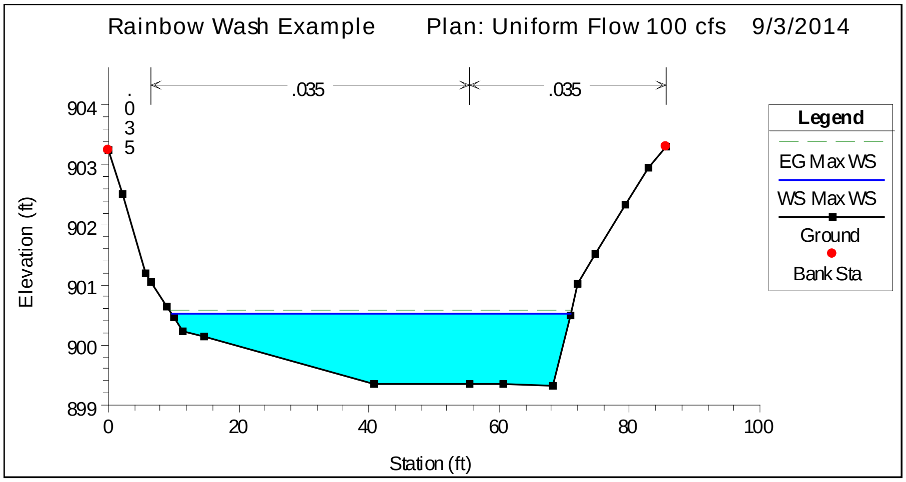
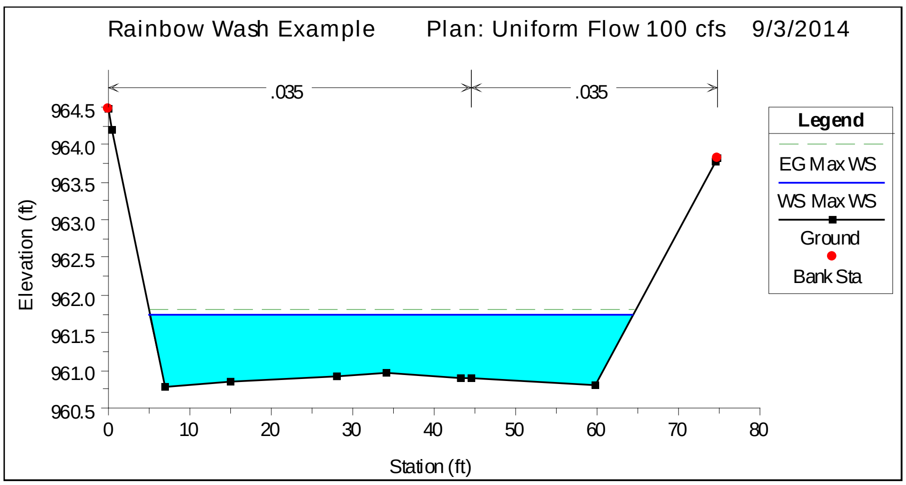


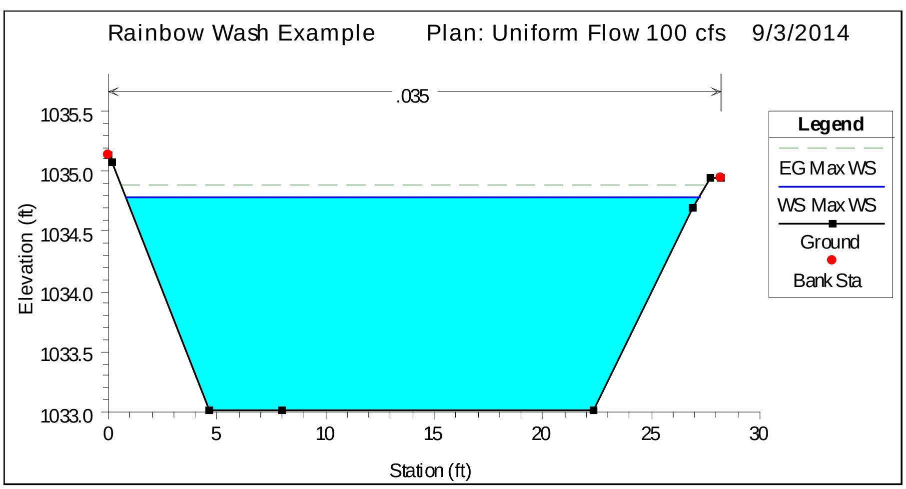
Figure 4.18 Representative Cross Sections



HEC-RAS Station 105.64, FLO-2D Grid 363080



HEC-RAS Station 13,677.4, FLO-2D Grid 506474



HEC-RAS Station 29,468.34, FLO-2D Grid 716953

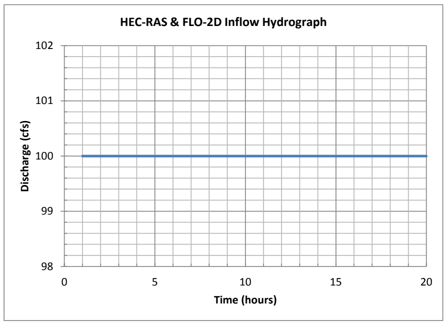
Figure 4.19 Rainbow Wash Channel in Vicinity of HEC-RAS Station 105.64

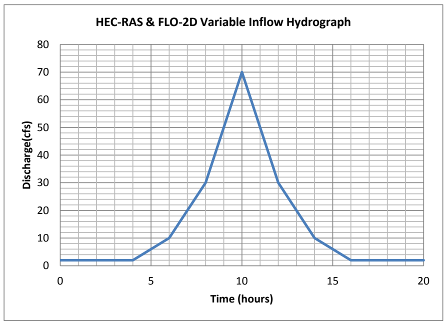
***HEC-RAS* Model Input Parameters**

The following is a summary of the key *HEC-RAS* model parameters:

1. Inflow: The model was used to compare two inflow scenarios; an unsteady uniform flow hydrograph of 100 cfs and an unsteady flow hydrograph with a peak discharge of 70 cfs. The inflow hydrographs are shown on Figure 4.20.
2. Boundary Conditions: The unsteady state *HEC-RAS* models used a flow hydrograph for the upstream condition and normal depth for the downstream condition.
3. Computation Parameters: The default parameters were used for options and tolerances.

Figure 4.20 *HEC-RAS* Model Inflow Hydrographs for Rainbow Wash





***FLO-2D* Model Input Parameters**

The base *FLO-2D* model that was used for this comparison came from the Gillespie *ADMS* project (Stantec Consulting Services, Inc., 2013). Refer to that study for detailed information on the base *FLO-2D* model and the topography. The *1D* channel for Rainbow Wash was added to that model. The *FLO-2D* model computational area and the Rainbow Wash *1D*-channel alignment are shown on Figure 4.21. The *FLO-2D* model primary control parameters were:

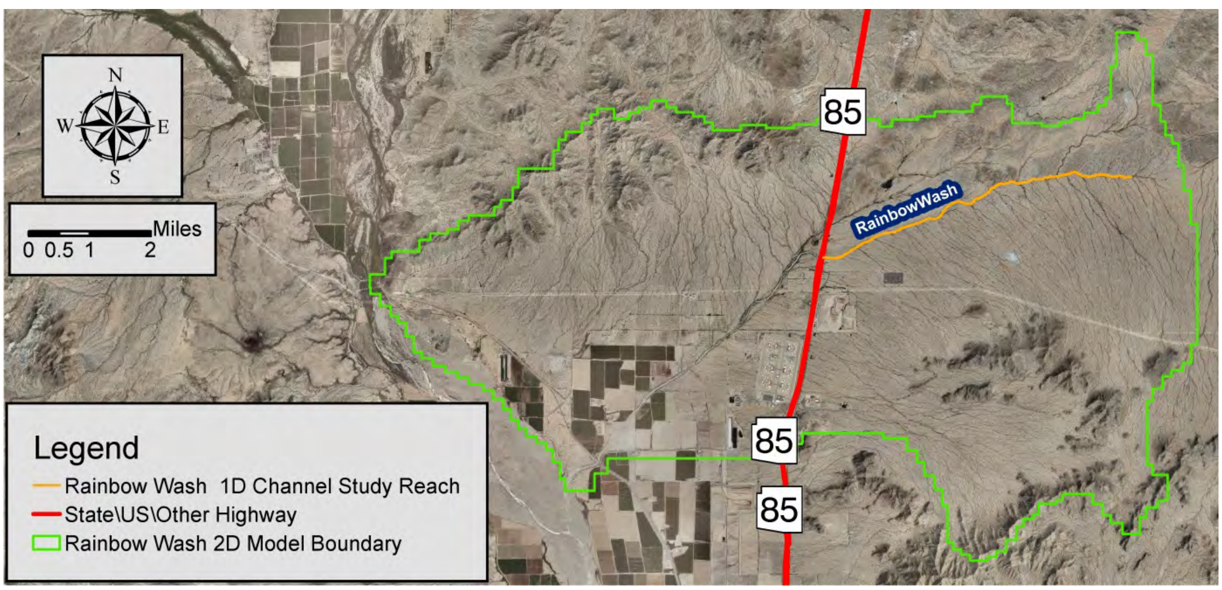
1. *1D* channel component applied.
2. No rainfall.
3. No infiltration.
4. No flow obstructions.
5. No hydraulic structures.
6. Inflow hydrographs: Same as used for the *HEC-RAS* models.
7. Depth variable *n*-values: off
8. Shallow *n*: off
9. Limiting *Froude number*: 0.90 (flow is subcritical)
10. Global *ARF*: 0
11. *TOL*: 0.01 feet
12. *1D* Channel Courant Number: 0.10
13. TIME_ACCEL: 0.10
14. *1D* Channel Cross Section Data: Identical to that used in the *HEC-RAS* models.
15. Model simulation time: 20 hours (same as for the *HEC-RAS* models)
16. Initial flow rate for the 70 cfs varying inflow hydrograph model: 0.6 cfs
17. Initial flow rate for the 100 cfs varying inflow hydrograph model: 1.7 cfs

Model Results

Both models were run for a 20-hour model duration. A global limiting *Froude Number* of 0.90 was applied, but no *n*-value adjustments resulted from this setting. This length of time allowed the upstream inflow to completely reach the end of the channel downstream. This was also helpful in keeping the *HEC-RAS* model stable under unsteady flow conditions. An unsteady *HEC-RAS* model needs an initial starting flow condition to estimate an initial stage to start flow computations.

Three cross sections were chosen to represent the results of the Rainbow Wash: the first upstream cross section (river station 29,468.34 at grid 716953), a cross section midway (river station 13,677.4 at grid 506474) and the second to last cross section at the downstream end (river station 105.64 at grid 363080). The water surface elevation results for the three cross sections for a constant flow rate of 100 cfs are shown on Figure 4.22. Note that the differences between the two models at the model boundaries are slightly greater than in the main reach.

Figure 4.21 2D Model Boundary



FLO-2D handles the boundary transitions differently than *HEC-RAS*. The differences are minor and acceptable to the *FCDMC*. The difference in depth in the main reach is very small, 0.03 feet. The *HEC-RAS* water surface elevation calculation tolerance setting for these models was 0.05 feet. *FLO-2D* provides results acceptable to *FCDMC* for this test case. The water surface elevation results for the three cross sections for a varying flow rate with a peak discharge of 70 cfs are shown on Figure 4.23. Again note that the differences between the two models at the model boundaries are slightly greater than in the main reach. The differences are minor and acceptable to the *FCDMC*. The difference in depth in the main reach at peak is 0.02 feet, which is smaller than for the uniform flow rate case. *FLO-2D* provides results acceptable to *FCDMC* for this test case.

The maximum water surface elevations were also compared for the entire 5.5 miles of Rainbow Wash. Over half of the *HEC-RAS* cross sections do not align with the center of the *FLO-2D* grid containing the cross section. However, the *HEC-RAS* maximum *WSEL* results for cross sections that are within 5 feet of the center of the *FLO-2D* grid were directly compared with the *FLO-2D* results for the uniform 100 cfs flow models. The average difference in *WSEL* was 0.007 feet. The average difference for all cross sections for the uniform 100 cfs flow models was 0.038 feet, including the boundary condition cross sections. The average difference for all cross sections for the varying inflow hydrograph models was 0.053 feet, including the boundary condition cross sections. Figure 4.24 and Figure 4.25 show the maximum *WSEL* results for the constant inflow hydrograph and varying inflow hydrograph test cases. *FLO-2D* provides results acceptable to *FCDMC* for this test case.

Figure 4.22 Comparison of *WSEL* Results for Constant Inflow at Selected Cross Sections

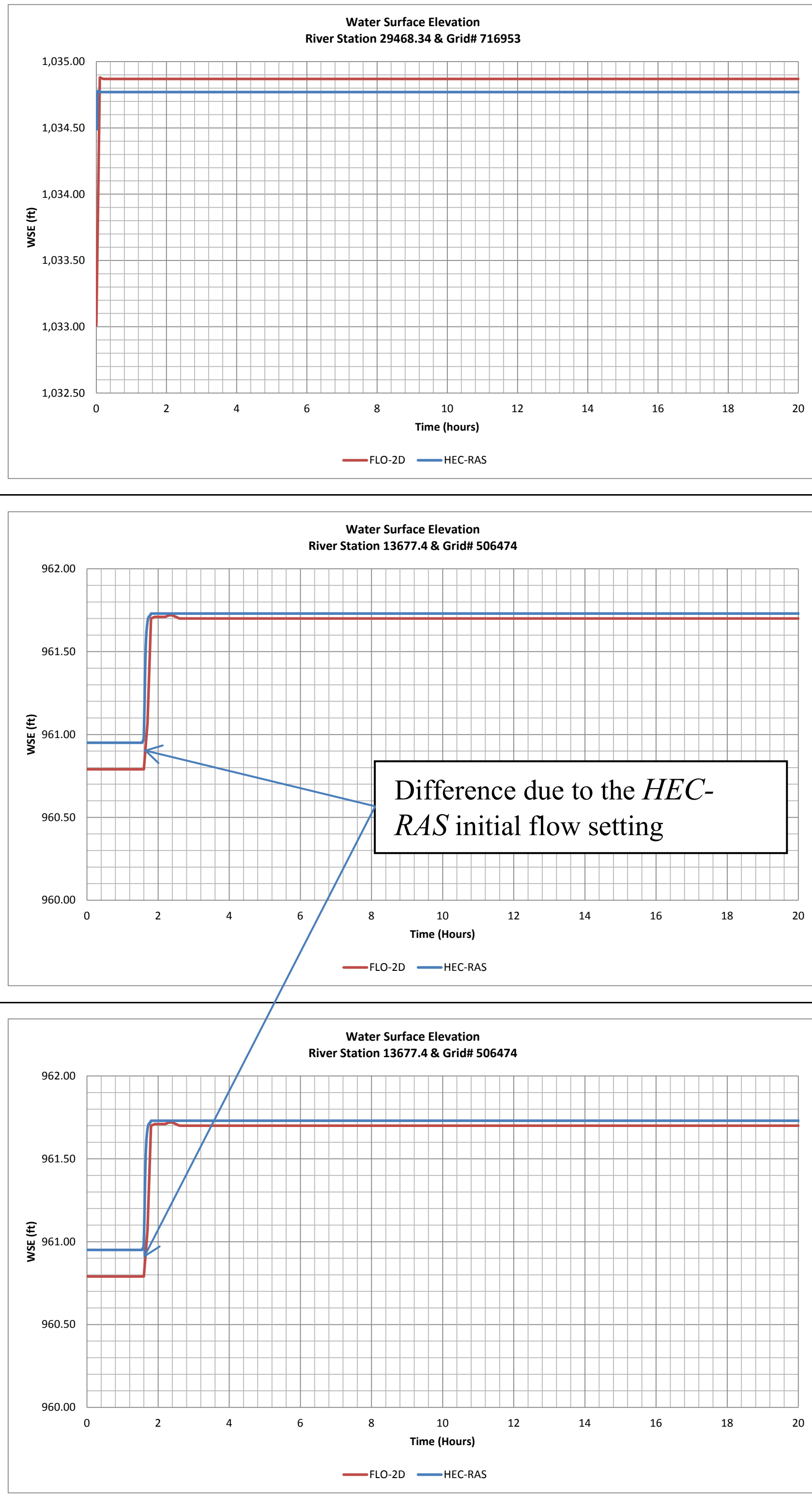
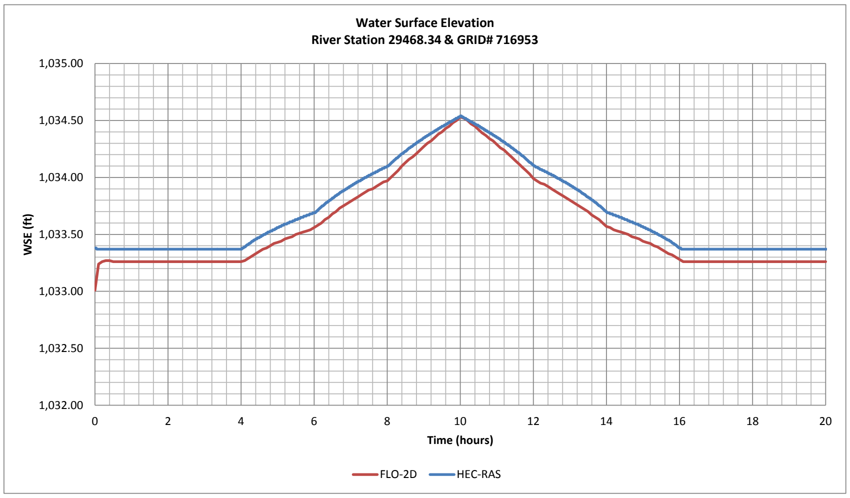
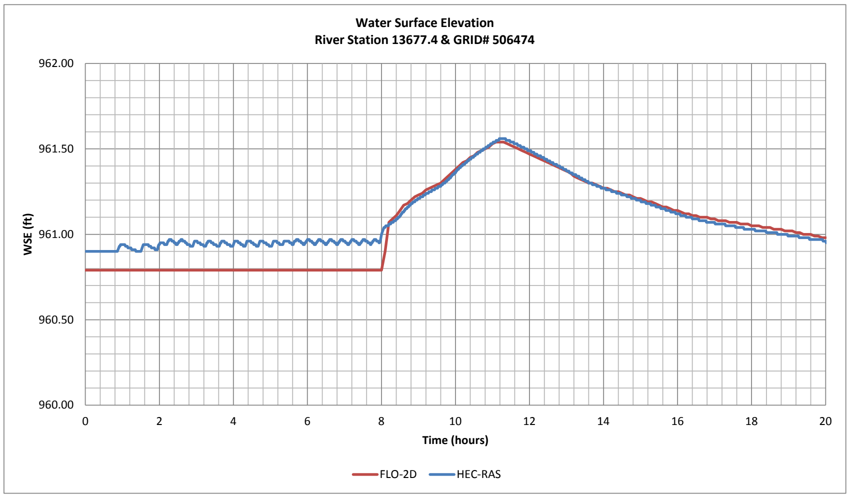


Figure 4.23 Comparison of WSEL Results for Varying Inflow at Selected Cross Sections





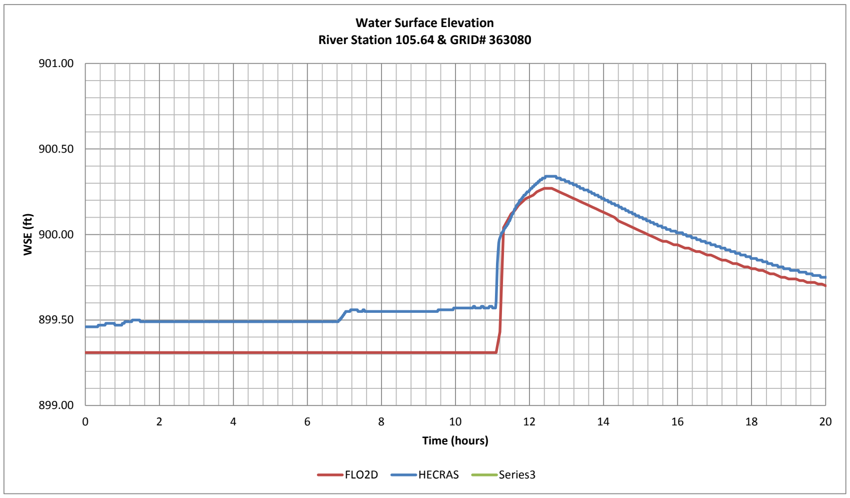
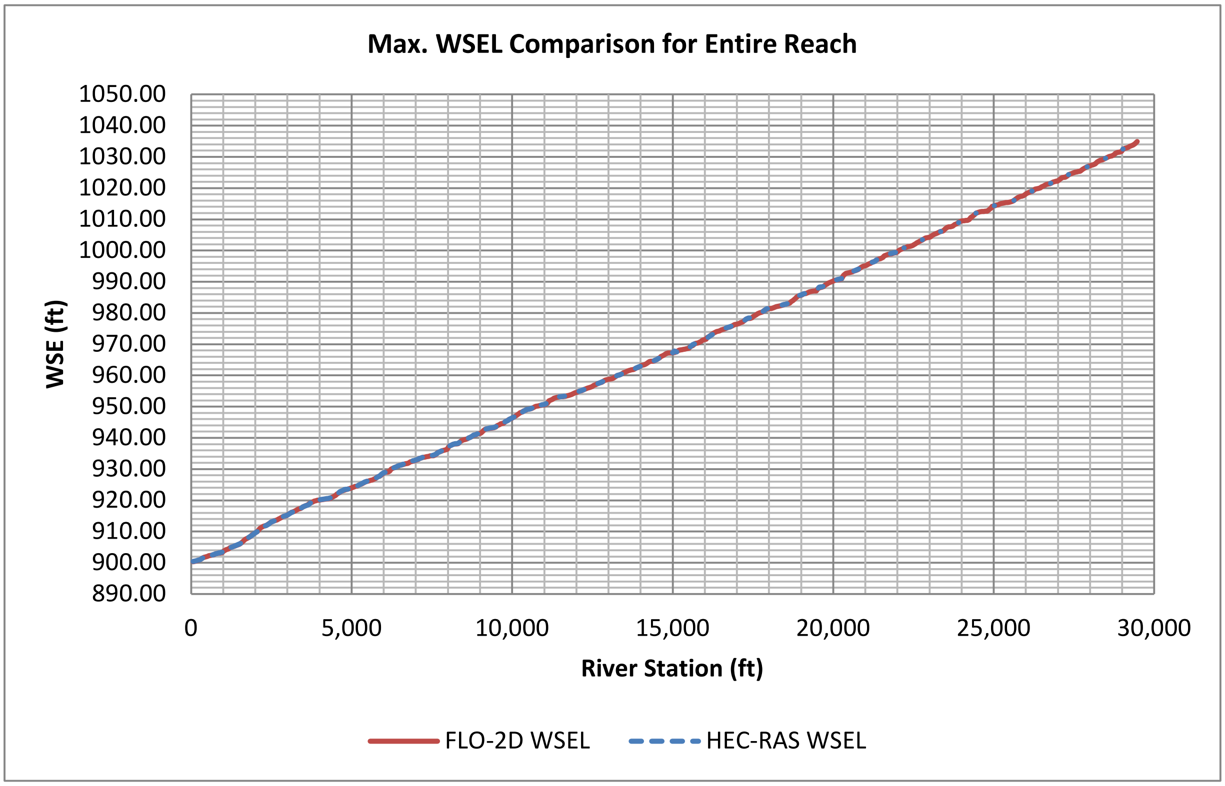


Figure 4.24 Maximum WSEL Results Comparisons for Uniform Inflow



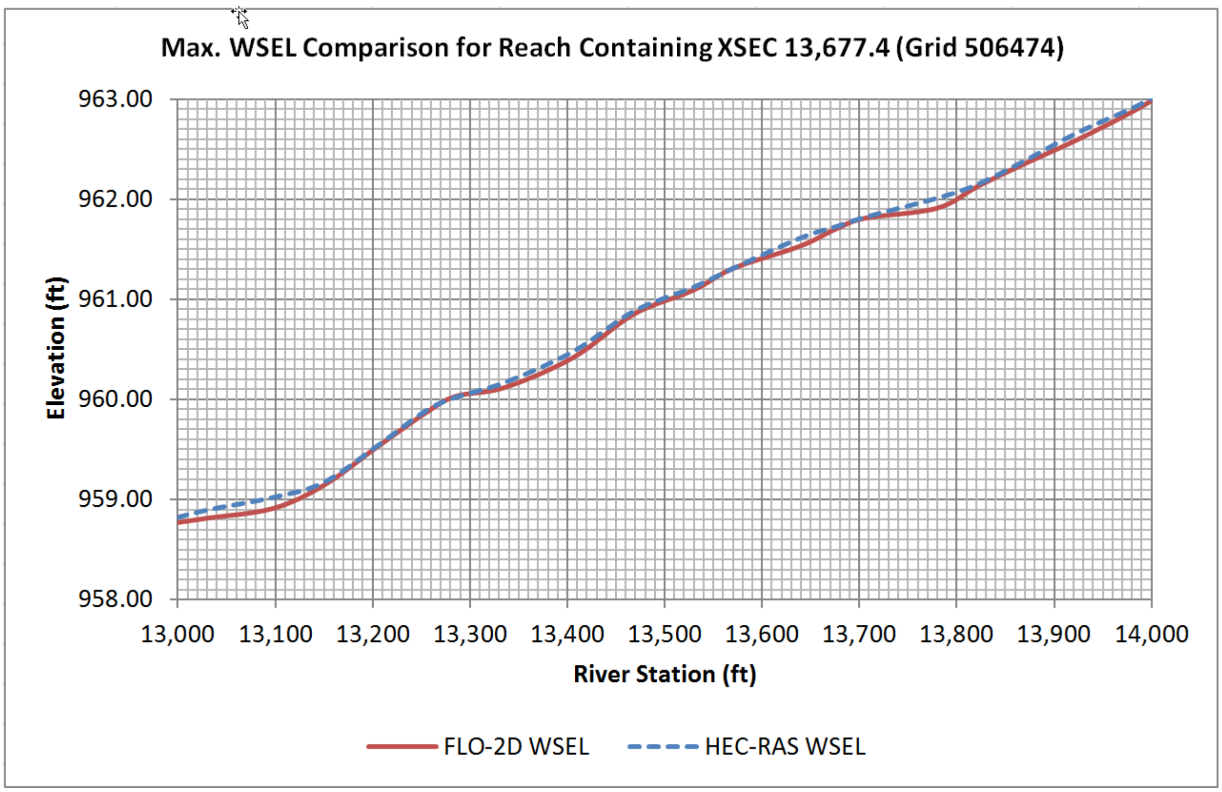
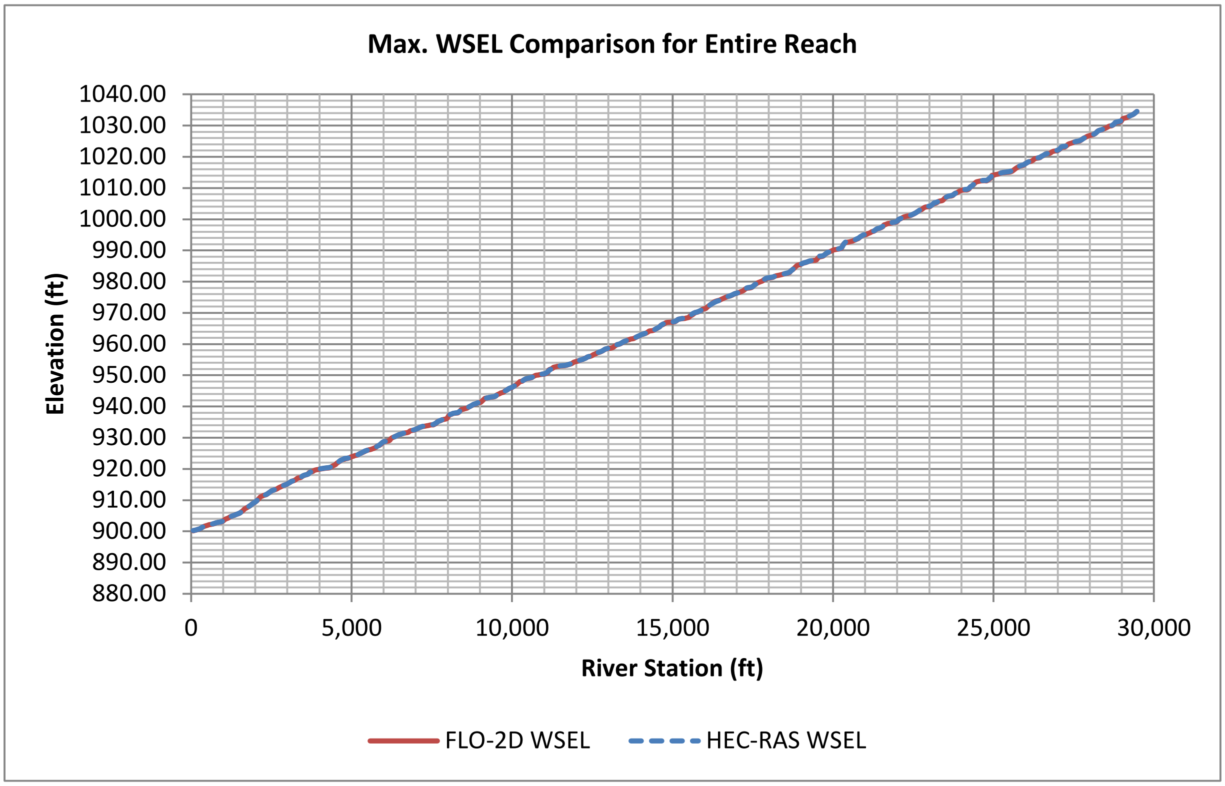
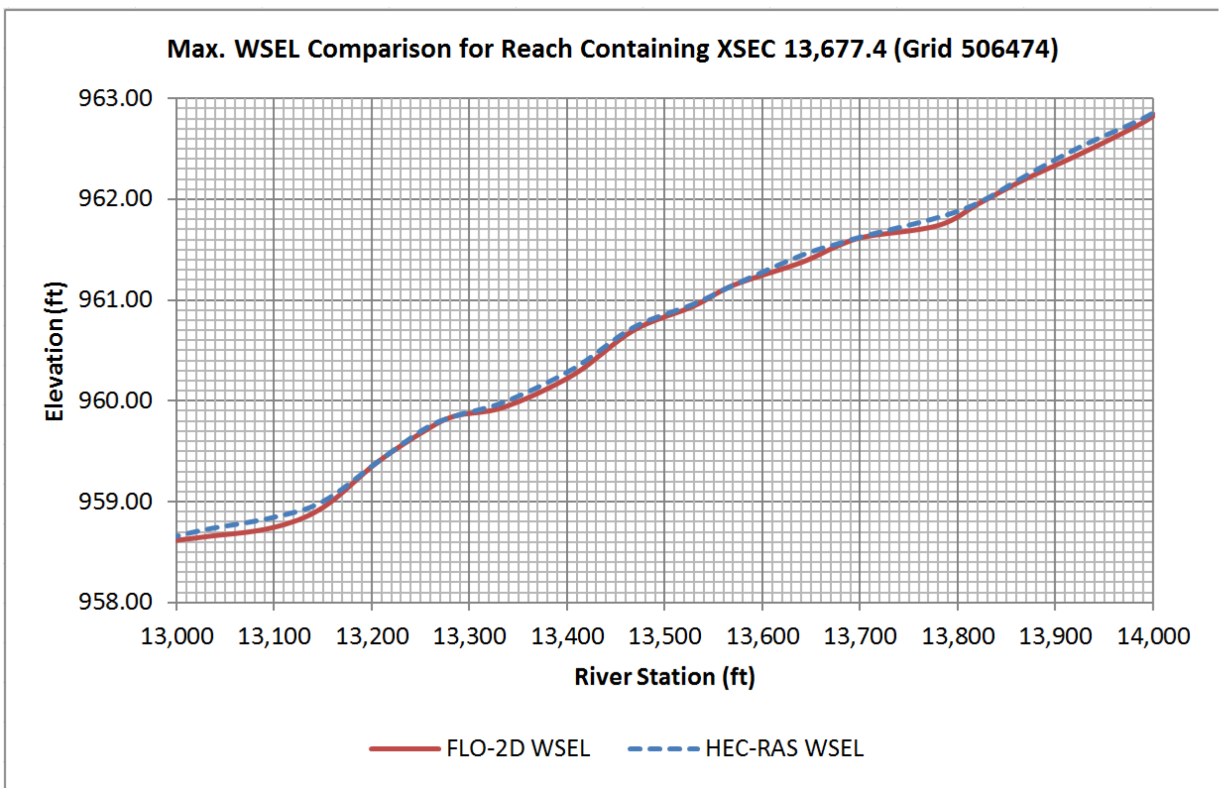


Figure 4.25 Maximum WSEL Results Comparisons for Varying Inflow





The routed volume results were compared at the three cross section locations to verify that the two models agree. *FLO-2D* does not report the total runoff volume by cross section so the routed hydrographs were used to calculate the total volume at each location. The volume results are shown in Table 4.12 and the plotted hydrographs for the varying flow condition are shown on Figure 4.26. Note that only the main flow portion of the hydrographs was used for comparison (after 8.0 hours for Midstream and after 11.1 hours for Downstream. Otherwise, the volume calculations would erroneously include the initial flow in *HEC-RAS* that is not included in *FLO-2D*.

Table 4.12 Comparison of Routed Volume Results for Rainbow Wash

	Volume (ac-ft)				Percent Difference
	FLO-2D	*HEC-RAS*			
Location	Hydrograph	Leading Edge	Hydrograph	Main Hydrograph (4)-(3)	[(5)-(2)]/(2)*100
(1)	(2)	(3)	(4)	(5)	(6)
Constant Inflow Hydrograph					
Upstream RS 29,468.34 (Grid 71695)	165.26	0.00	165.29	165.29	0.02
Midstream RS 13,677.4 (Grid 506474)	151.21	0.00	152.09	152.09	0.58
Downstream RS 105.64 (Grid 363080)	137.63	0.00	138.31	138.31	0.49
Variable Inflow Hydrograph					
Upstream RS 29,468.34 (Grid 71695)	26.43	0.00	26.45	26.45	0.08
Midstream RS 13,677.4 (Grid 506474)	25.07	1.16	26.27	25.11	0.16

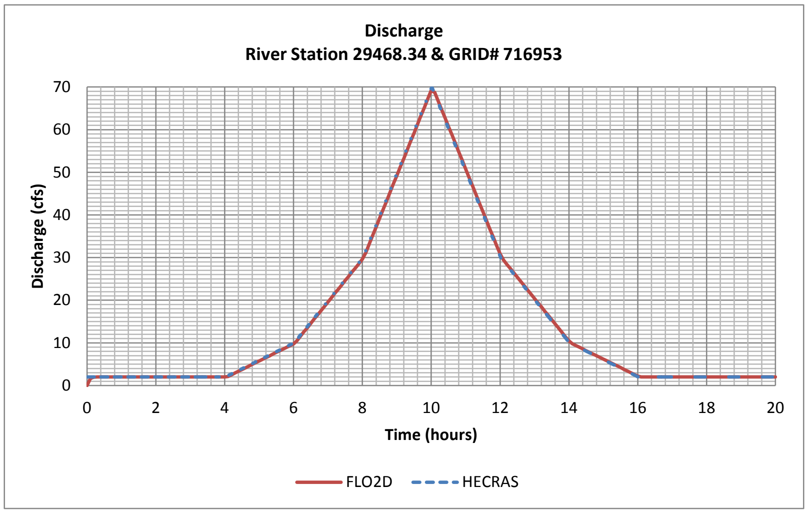
Table 4.12 Comparison of Routed Volume Results for Rainbow Wash					
	Volume (ac-ft)				**Percent Difference**
	FLO-2D	***HEC-RAS***			
Location	**Hydrograph**	**Leading Edge**	**Hydrograph**	**Main Hydrograph (4)-(3)**	**[(5)-(2)]/(2)*100**
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**
Downstream RS 105.64 (Grid 363080)	23.00	1.17	24.79	23.62	2.70

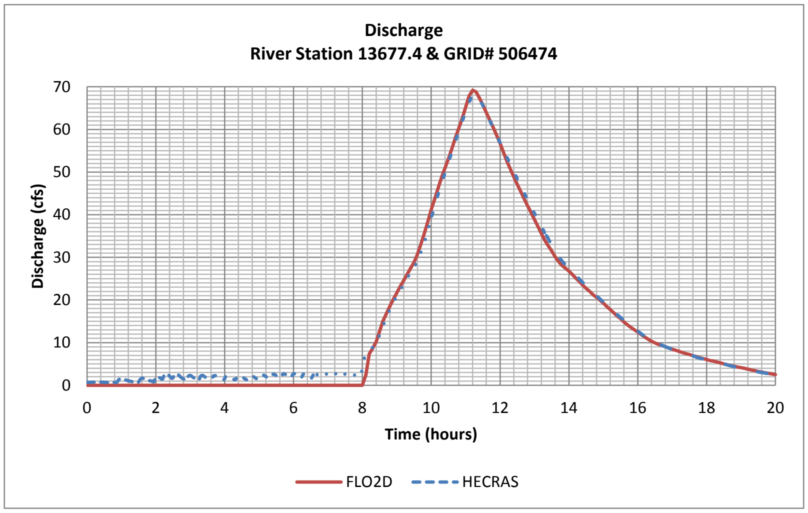
The hydrograph volume differences between *FLO-2D* and *HEC-RAS* at the three cross section locations shown in Table 4.12 are minimal. The inflow and routed hydrograph comparisons shown on Figure 4.26 are nearly identical. *FLO-2D* provides results acceptable to *FCDMC* for the volume and routed hydrograph check.

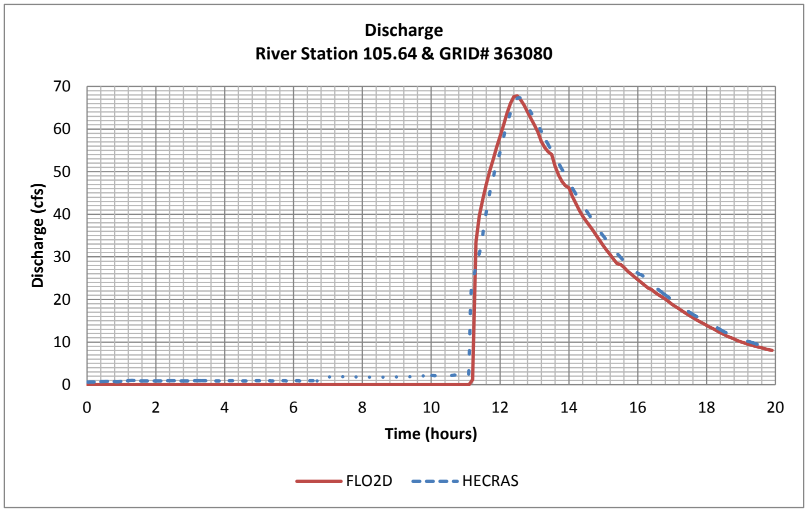
<u>Conclusion</u>

The Rainbow Wash *1D* natural channel test case shows reasonably comparable results with *HEC-RAS*. *FLO-2D* provides results acceptable to *FCDMC* for application of the *1D* channel routine for natural washes.

Figure 4.26 Discharge Hydrograph Results Comparisons for Varying Inflow







4.2.3 Flow Obstructions

4.2.3.1 General

Flow obstructions are simulated using *ARF* factors, *WRF* factors, or a combination of the two as described in Section 2.4.6. The floodplain grid flume model described in Section 4.2.2.2 was used as the base for preparing *FLO-2D* test models. The following flow obstruction scenarios were tested to verify that *FLO-2D* applies these factors appropriately:

1. Single Grid – *ARF*: A single grid with *ARF* = 1 applied using blockage method 1 ("T" records)
2. Multiple Grids – *ARF*: A block of four grids with *ARF* = 1 applied using blockage method 1
3. Single Grid – *WRF*: A single grid with all sides blocked using *WRF* = 1
4. Multiple Grids – *WRF*: A block of four grids with all external sides blocked using *WRF* = 1
5. Single Side Open: All flow forced through a single grid with one side fully open
6. Single Side Partially Blocked: All flow forced through a single grid with one side partially blocked

4.2.3.2 Scenario 1: Single Grid – *ARF*

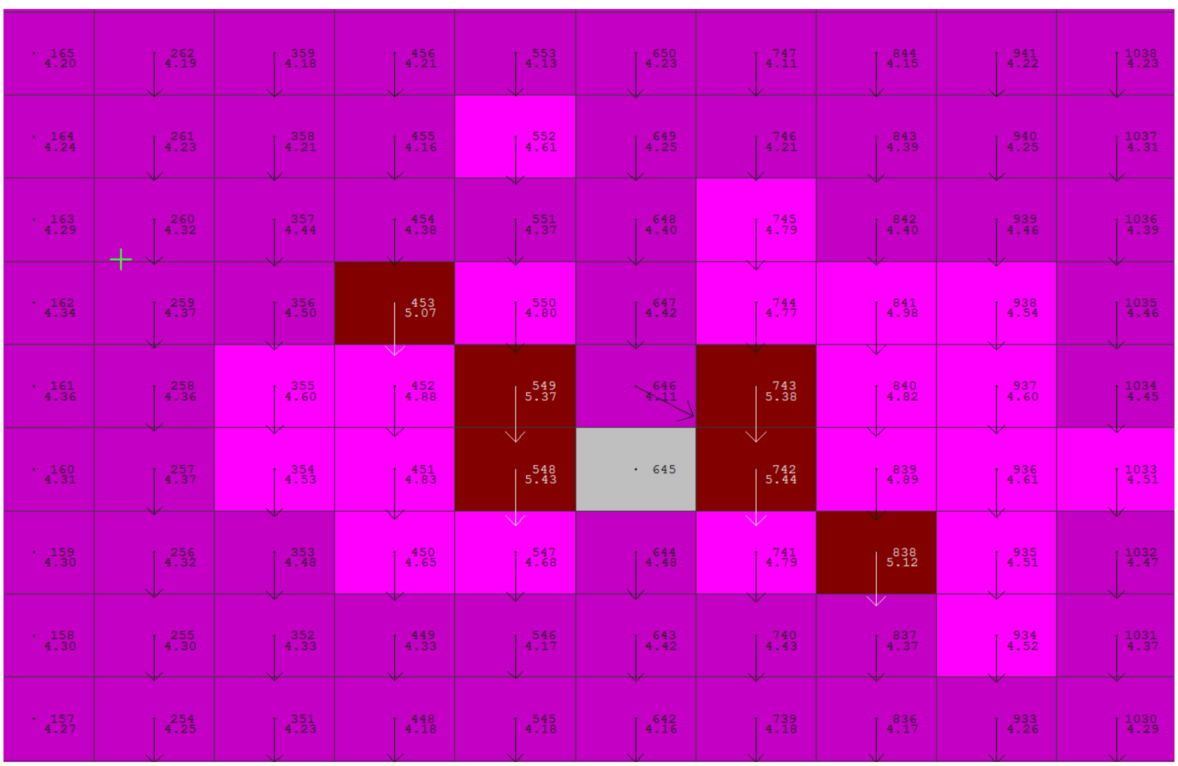
This scenario tests the blockage of a single grid element using an *ARF* of 1.0. Grid 645 was blocked. The total discharge imposed at the upstream end of the model was 1,500 cfs. The flow depth and velocity results are shown on Figure 4.27. Note that there is no flow depth or velocity reported for Grid 645 (shaded grey). The single *ARF* obstruction function works as expected.

4.2.3.3 Scenario 2: Multiple Grids – *ARF*

This scenario tests the blockage of a group of four grid elements using an *ARF* of 1.0. Grids 547, 548, 644 and 645 were blocked (shaded grey). The total discharge imposed at the upstream end of the model was 1,500 cfs. The flow depth and velocity results are shown on Figure 4.28. Note that there is no flow depth or velocity reported for the four blocked grids. Also note the increase in flow depth and velocity as flow is diverted around the blocked grids. The multiple *ARF* obstruction function works as expected.

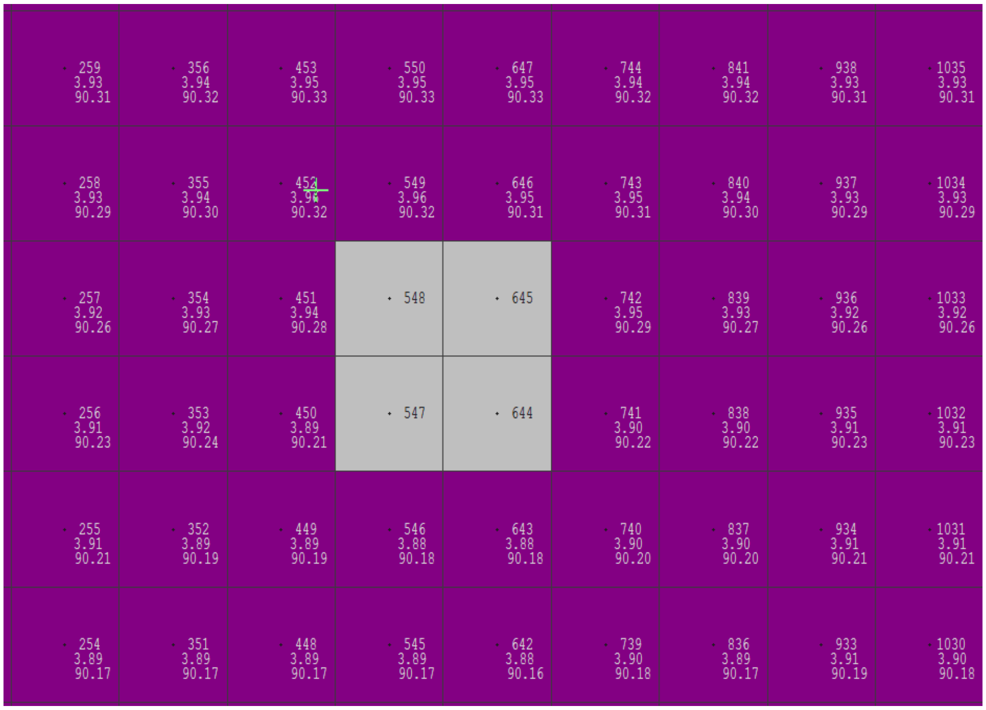
Figure 4.27 Results of Single Grid Obstruction using *ARF*

Grid Number, Depth, and *WSEL*

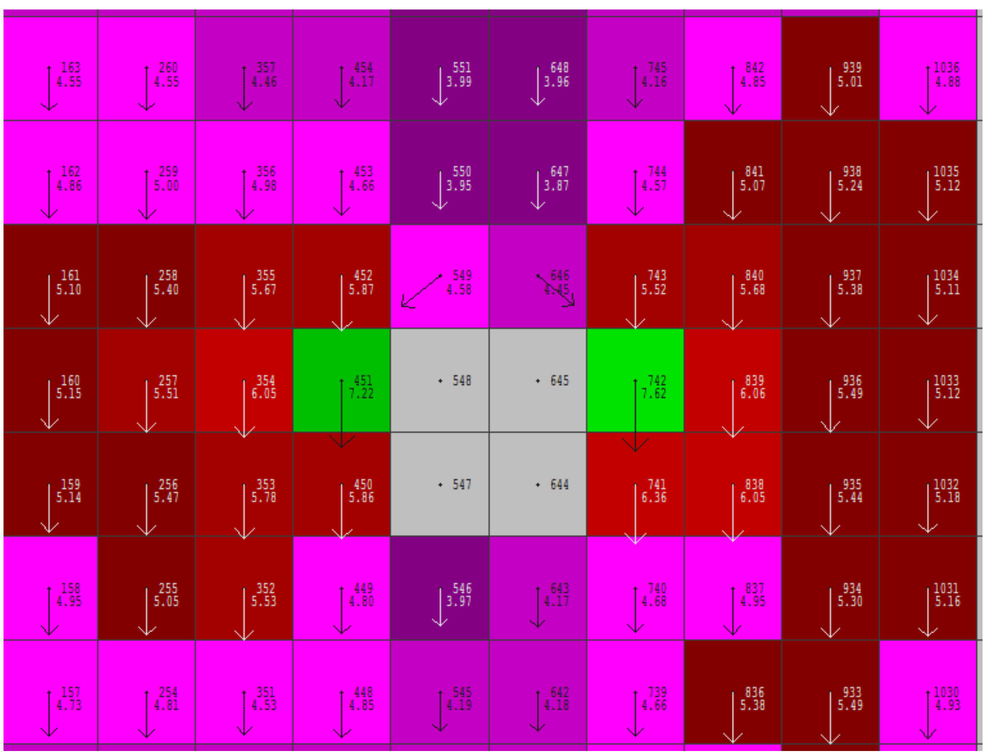


Grid Number, Velocity and Vector

Figure 4.28 Results of Multiple Grid Obstruction using *ARF*



Grid Number, Depth, and *WSEL*



Grid Number, Velocity and Vector

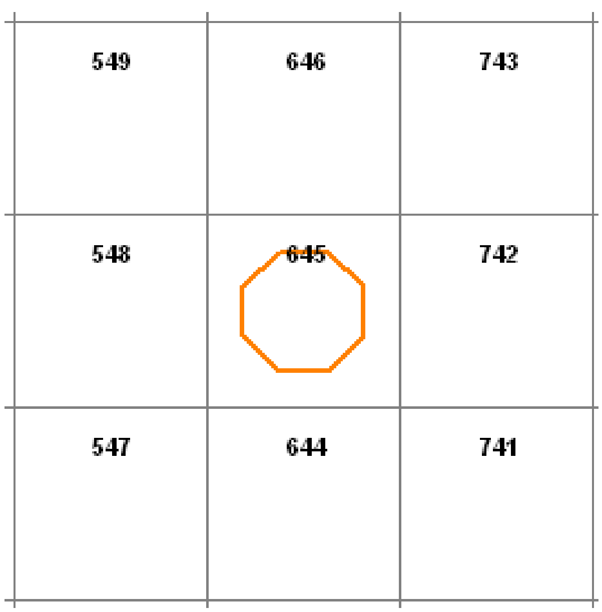
4.2.3.4 Scenario 3: Single Grid – W*RF*

This scenario tests the blockage of a single grid element using a *WRF* of 1.0 for all eight sides as shown in the left example of Figure 4.29. Grid 645 was blocked. The total discharge imposed at the upstream end of the model was 1,500 cfs. The flow depth and velocity results are shown on Figure 4.30. Note that there is no flow depth or velocity reported for Grid 645 and that the results for adjacent grids match those from Section 4.2.3.2. The single *WRF* obstruction function works as expected.

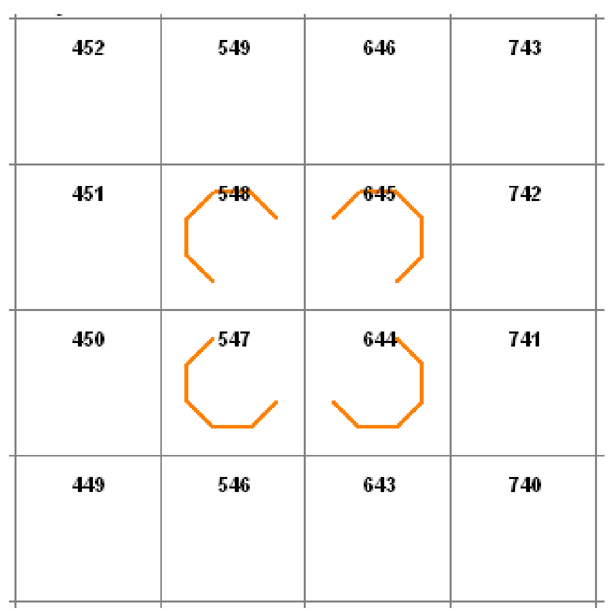
4.2.3.5 Scenario 4: Multiple Grids – W*RF*

This scenario tests the blockage of a group of four grid elements using a *WRF* of 1.0. Grids 547, 548, 644 and 645 were blocked as shown in the right side of Figure 4.29. The total discharge imposed at the upstream end of the model was 1,500 cfs. The flow depth and velocity results are shown on Figure 4.31. Note that there is no flow depth or velocity reported for the four blocked grids (shaded grey). Also note the increase in flow depth and velocity as flow is diverted around the blocked grids. These results also closely match those described in Section 4.2.3.3. The multiple grid *WRF* obstruction function works as expected.

Figure 4.29 Single and Multiple Grids Blockage Schematic

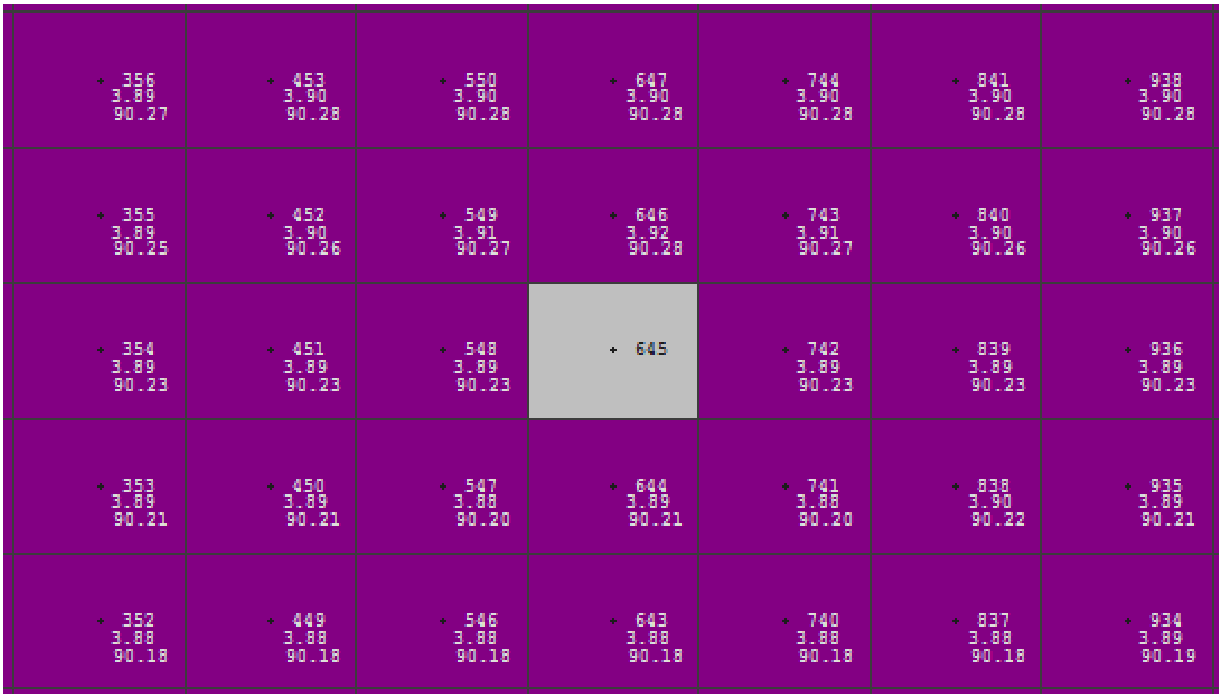


Single Grid - *WRF*

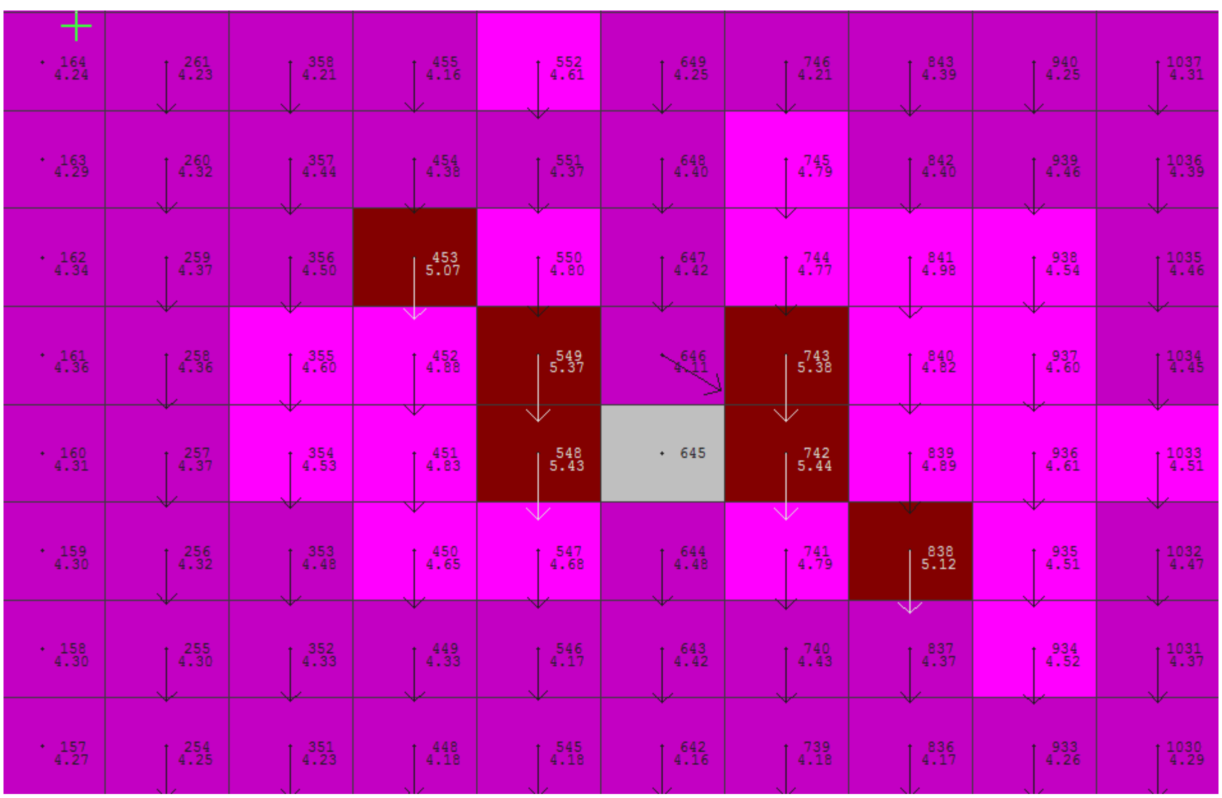


Multiple Grid - *WRF*

Figure 4.30 Results of Single Grid Obstruction using *WRF*

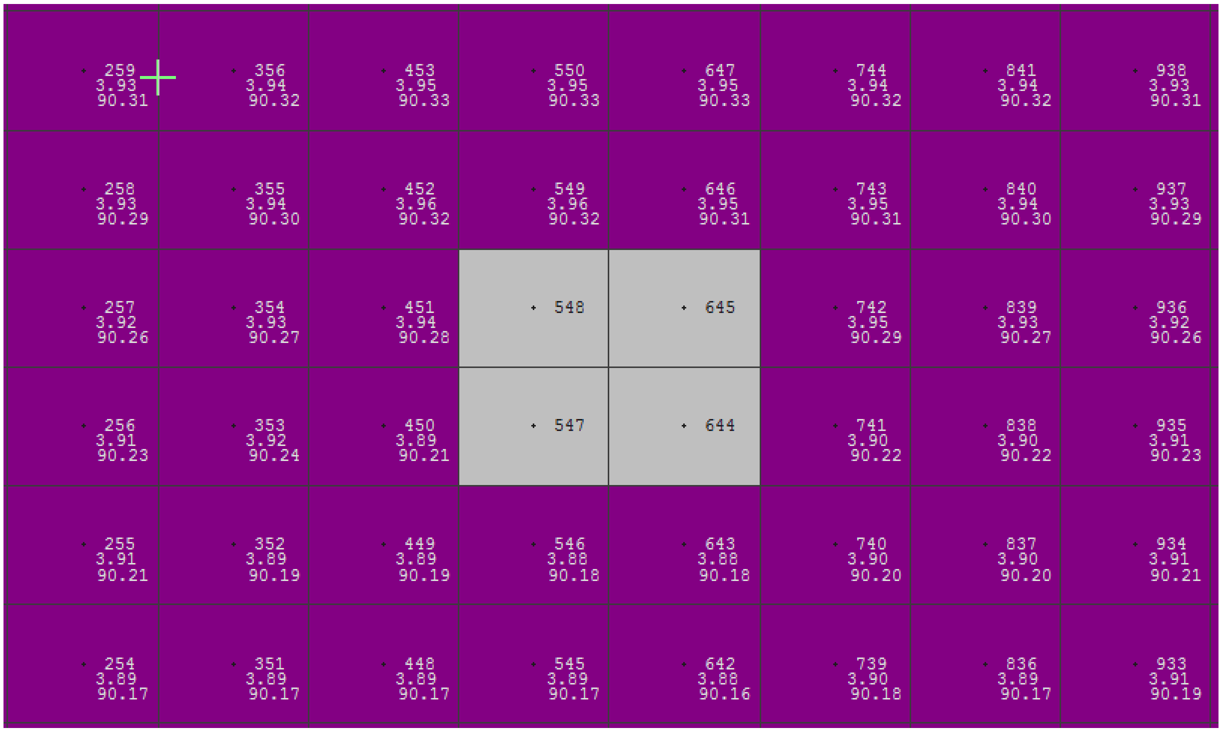


Grid Number, Depth, and WSEL

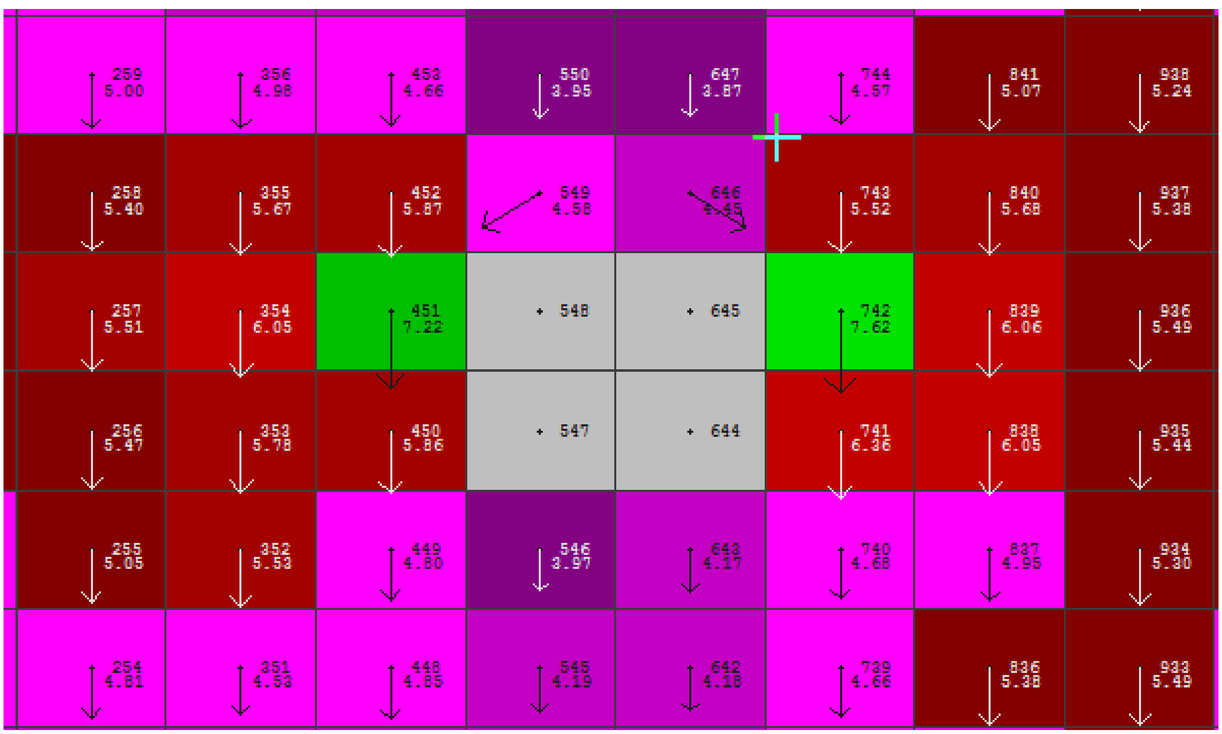


Grid Number, Velocity and Vector

Figure 4.31 Results of Multiple Grid Obstruction using *WRF*



Grid Number, Depth, and *WSEL*



Grid Number, Velocity and Vector

4.2.3.6 Scenario 5: Single Side Open

This scenario tests flow through a grid that has only one open side (Grid 645) as shown on Figure 4.32. The same model used for the previous four scenarios was modified as follows:

1. The total inflow discharge at the upstream end was changed to 50 cfs.
2. The grids in the same row as Grid 645 (shaded grey) were totally blocked using an *ARF* of 1.0.
3. The top of Grid 645 was blocked except for side 1 (North), which was set to a WRF of 0.0.
4. Grids 545-547 and 739-741 were completely blocked using an *ARF* of 1.0 to force flow from Grid 645 due south. This grid blockage configuration is shown on Figure 4.32.
5. Note that for Grids 643-644, *FLO-2D* only uses sides 1 (north) and 3 (south) for conveyance. Because the adjacent grids are blocked, sides 5, 2, 6, 7, 4, and 8 of Grids 642-645 are blocked by default even though they are not coded as blocked in the *ARF*.DAT input data file. Grid 642 can share flow with Grids 544, 641, and 738 through sides 3, 6, and 7.

The results for maximum depth and velocity are shown on Figure 4.33. Refer to Figure 4.9 for a diagram of the *FLO-2D* grid 8-sided representation. Note that all flow is directed to Grid 645 and then routed due south through a single column of three grids before being allowed to spread back out across the flume. A simple hydraulic check of the results between grids 645 and 644 is shown in Table 4.13: From this check, flow patterns in and out of Grid 645 are as expected, and the manual results closely match those reported by *FLO-2D*.

Table 4.13 Hydraulic Calculations Check for Single Side Open Model

Grid	Time	Elev	Depth	*WSEL*	Hydraulic Gradeline, S_{hg}	Area, *A*	Wetted Perimeter, *P*	Hydraulic Radius, *R*
	hrs	ft	ft	ft	ft/ft	sf	ft	ft
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
645	1.08	86.34	1.826	88.166		7.563	4.142	1.826
644	1.08	86.32	1.685	88.005	0.0161	6.979	4.142	1.685

$$S_{hg} = \frac{88.166-88.005}{10} = 0.0161\ ft/ft \qquad A_{avg} = \frac{7.563+6.979}{2} = 7.271\ sf$$

$$Q = VA_{avg} = 6.860 * 7.271 = 49.88\ cfs\ (V \text{ is from TIMDEP.OUT})$$

The maximum velocity reported in VELFP.OUT for Grid 645 was 6.84 fps.

The discharge reported in MAXQBYDIR.OUT for Grid 645 at time 1.08 hrs was 49.81 cfs.

Figure 4.32 Grid Blockage Schematic for Single Grid Side Opening

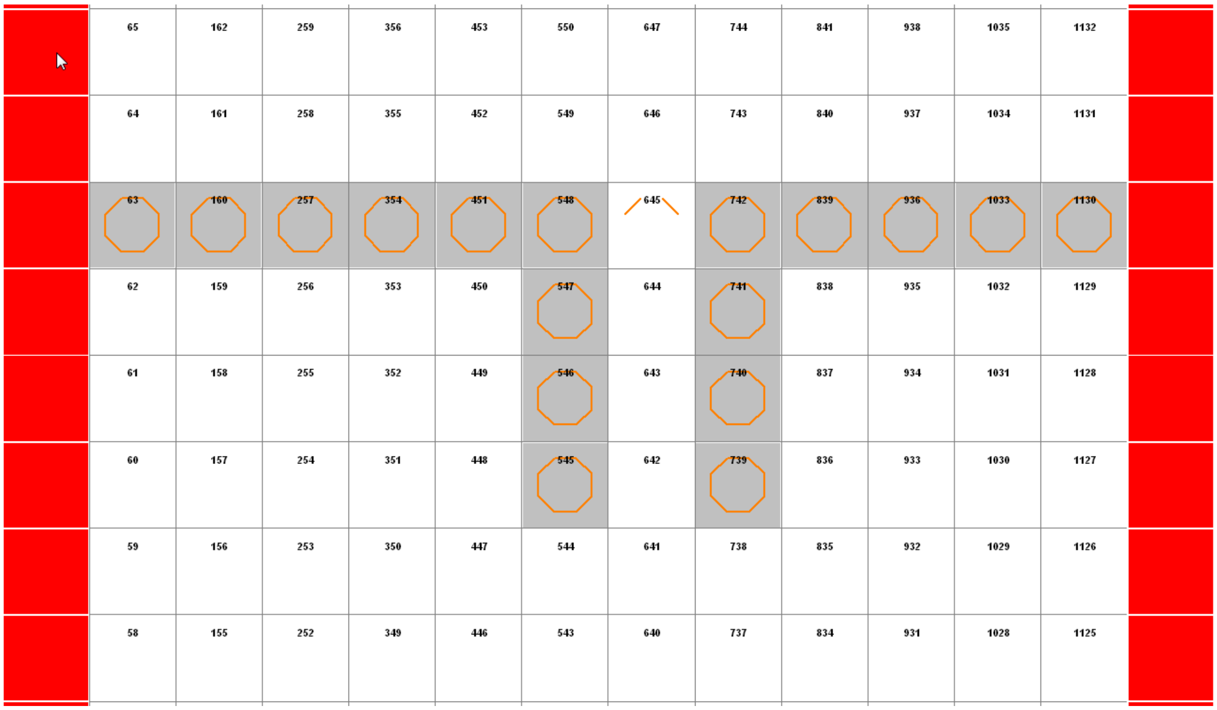
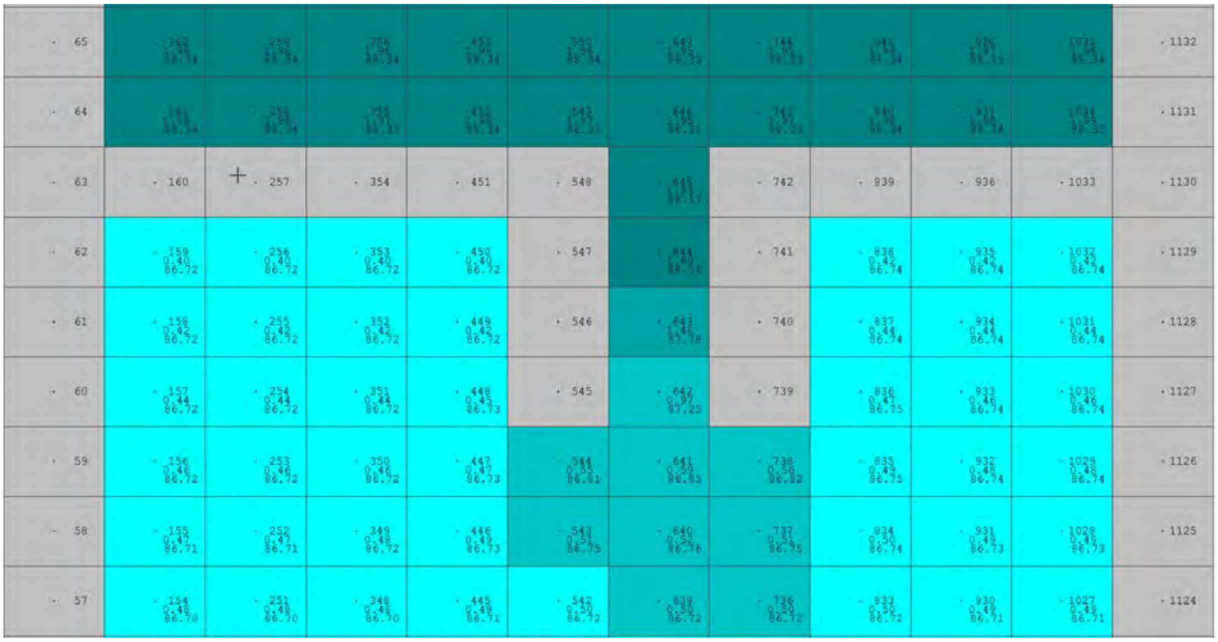
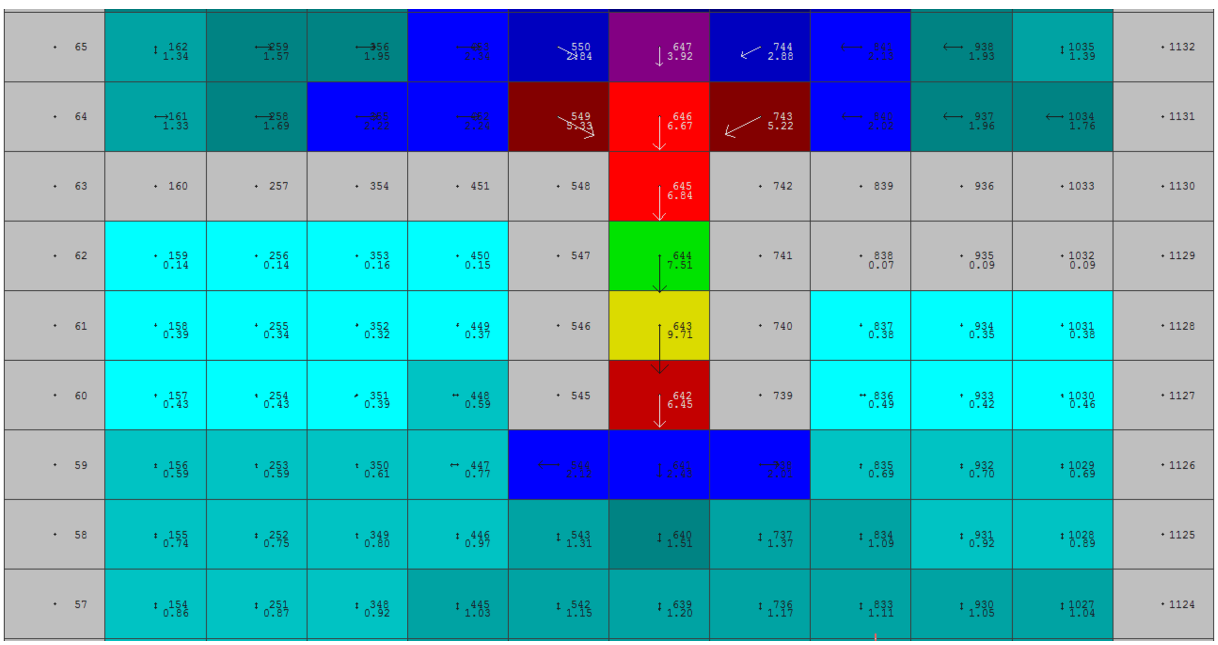


Figure 4.33 Results for Single Grid Side Opening



Grid Number, Depth, and *WSEL*



Grid Number, Velocity and Vector

4.2.3.7 Scenario 6: Single Side Partially Open

For this scenario, the Scenario 5 Single Side Open model was revised, setting the *WRF* value for Grid 645 Side 1 (north) to 0.50.

Using data from the MAXQHYD.OUT file for the run, and evaluating flow from Grid 646 to Grid 645:

Table 4.14 Hydraulic Calculations Check for Single Side Partially Blocked Model

	Time	**Elev**	**Depth**	***WSEL***	**Hydraulic Gradeline, S_{hg}**	**Area, *A***	**Wetted Perimeter, *P***	**Hydraulic Radius, *R***
Grid	**hrs**	**ft**	**ft**	**ft**	**ft/ft**	**sf**	**ft**	**ft**
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**
646	1.07	86.36	2.18	88.54		4.51	2.071	2.18
645	1.07	86.34	1.79	88.13	0.041	3.71	2.071	1.79

$$S_{hg} = \frac{88.54 - 88.13}{10} = 0.041\ ft/ft$$

$$A_{avg} = \frac{4.51 + 3.71}{2} = 4.11\ sf$$

$$R = \frac{A}{P};\ Average\ R = (2.18 + 1.79)/2 = 1.985\ ft$$

$$V = (1.486R^{2/3}S_{hg}^{0.5})/n = (1.486 * 1.985^{0.667} * 0.041^{0.5})/0.040 = 11.881\ fps$$

$$Q = VA_{avg} = 11.881 * 4.11 = 48.83\ cfs$$

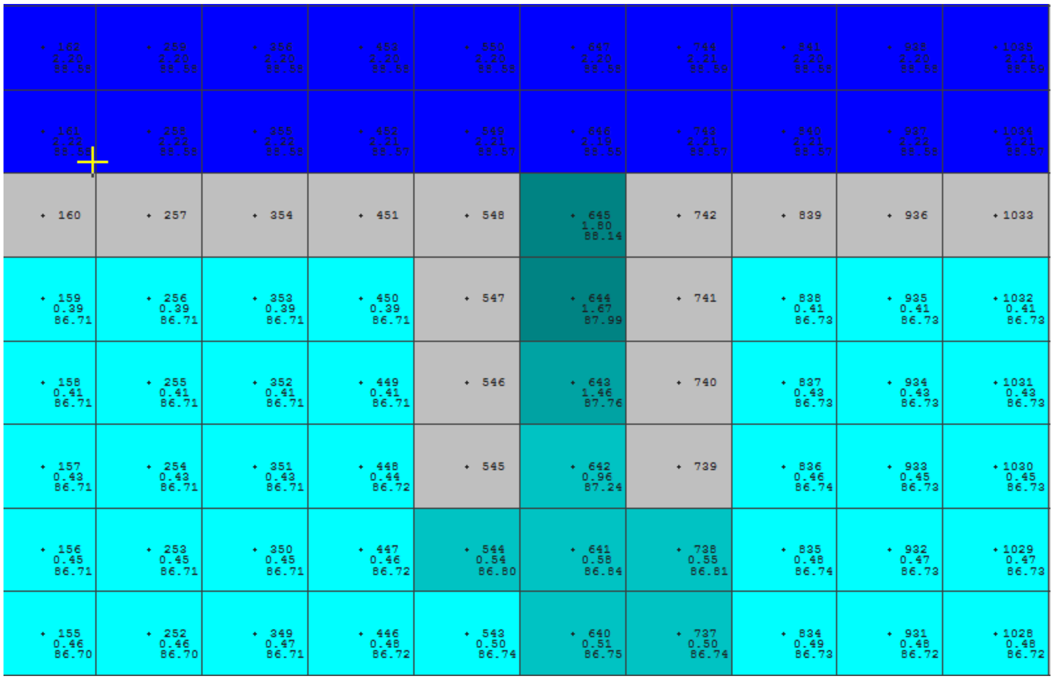
The maximum velocity reported in VELFP.OUT for Grid 646 was 11.88 fps

The discharge reported in MAXQBYDIR.OUT for Grid 646 at time 1.07 hrs was 48.63 cfs.

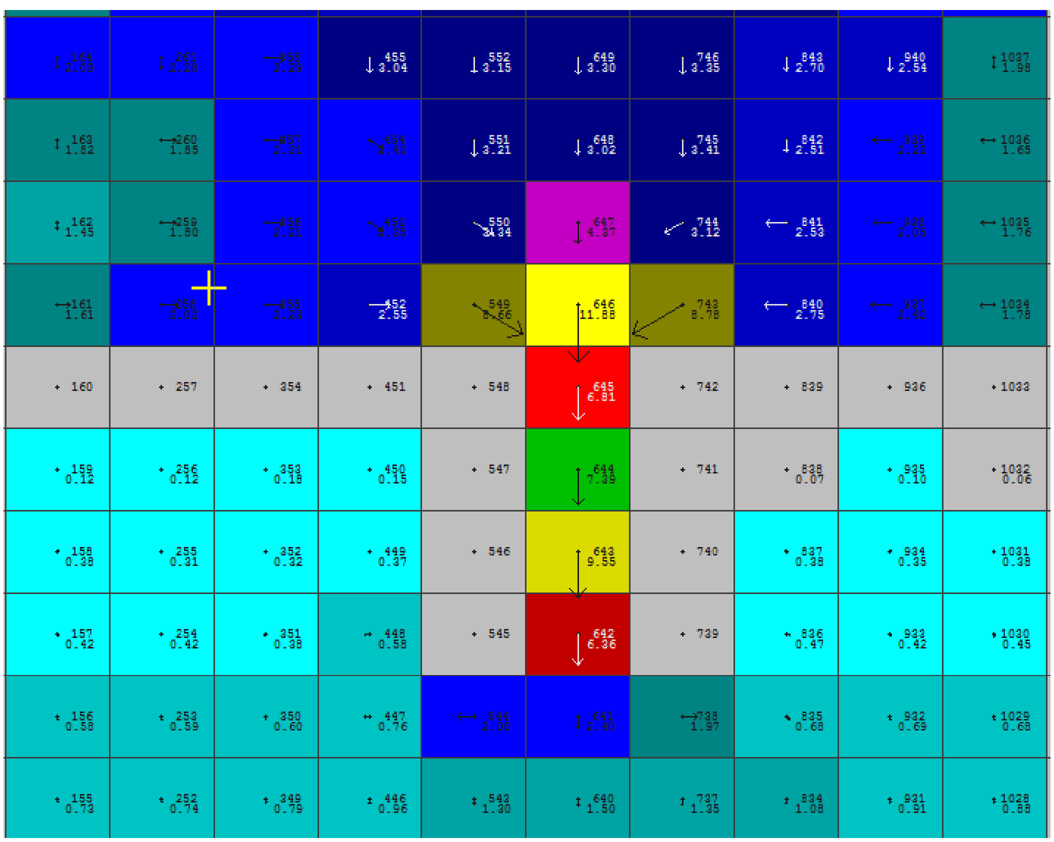
The flow patterns approaching and leaving Grid 645 are as expected. Note that the depths upstream of the 50% constriction at Grid 645 increased as compared with the un-constricted run. The velocity into and out of Grid 645 also increased as expected. The results from this manual check closely match those reported by *FLO-2D*.

Based on these six scenarios, the *FLO-2D* flow obstruction methods function appropriately and are acceptable for *FCDMC* modeling purposes.

Figure 4.34 Results for Single Grid Side Partial Opening



Grid Number, Depth, and *WSEL*



Grid Number, Velocity and Vector

4.2.4 Levees

4.2.4.1 General

The purpose of this section is to verify that hydraulic calculations performed by *FLO-2D* for levees are done appropriately by comparing the *FLO-2D* results with results from the *HEC-RAS* computer program. Verification models were prepared for the following scenarios:

1. Flume Model – A flume type channel simulated with levees for the sides.
2. Levee Breach – A flume type channel with a levee breach.

4.2.4.2 Scenario 1: Flume Model

The purpose of this test is to verify that hydraulic calculations for the model using levees are performed appropriately. This test consists of a rectangular flume with the following characteristics:

1. Base Width: 14.142 feet for *HEC-RAS*, 20 feet for *FLO-2D* (see discussion below)
2. Depth: 10 feet
3. Slope: uniform at 2%
4. *n*-value: 0.040
5. Length: 200 feet

The flow regime is sub-critical. A *HEC-RAS* model with the above characteristics was built using cross sections at 10 foot intervals and a uniform peak discharge of 140 cfs. A lateral structure is included on the right side, which will be used in Scenario 2 to model a levee break. A *3D* view of the *HEC-RAS* flume for 140 cfs is shown on Figure 4.35. This model has a base width of 14.142 feet to make a fair comparison to the *FLO-2D* model. Further discussion regarding the wetted perimeter difference between *FLO-2D* and *HEC-RAS* is included in Section 4.2.2.2. The results of this model supply the benchmark for testing *FLO-2D*.

A *FLO-2D* model was built using 3,618 uniform square 10-foot grid elements. The flume was modeled by assigning a levee between grid numbers 2010 and 1898 and between 2211 and 2099. The area downstream of the levee was modeled by insetting a rectangular channel simulated with two grid elements forming the base, 10 feet below the grade of the adjacent elements as shown on Figure 4.36.

In order to be able to directly compare the *FLO-2D* results with *HEC-RAS*, the depth variable *n* method in *FLO-2D* must be turned off. The limiting Froude number was set to 0.90 to force sub-critical flow because HEC-RAS was run in sub-critical mode. Inflow to the *FLO-2D* model was spread out evenly over two grids by dividing the total inflow ordinates by two and applying the resulting hydrograph to each of the two grid elements. Important *FLO-2D* input control settings used were:

1. Limiting Froude number = 0.90 to force sub-critical flow
2. Shallow *n* = 0.00
3. AMANN = -99
4. Floodplain COURANTFP = 0.1
5. *TOL* = 0.004

The results from the *FLO-2D* and *HEC-RAS* models are compared in Table 4.15. The average depth, velocity and area results for selected *HEC-RAS* cross sections and the corresponding *FLO-2D* grids are shown on this table. The results are comparable but slightly different between the two models. Slight differences are to be expected because the computation methods between a *1D* model and a grid-based *2D* model are dissimilar, and *FLO-2D* uses a different wetted perimeter than *HEC-RAS*. As an additional check, the velocity and area values were used to compute the discharges shown in the last columns. The results are reasonable and acceptable for *FCDMC* purposes.

The conclusion is that *FLO-2D* levee method performs hydraulic calculations in an acceptable manner. If a channel is to be modeled using a *FLO-2D* levee, the lack of conveyance of a portion of each bank grid element should be accounted for by the modeler.

Figure 4.35 Levee Model Using *HEC-RAS*

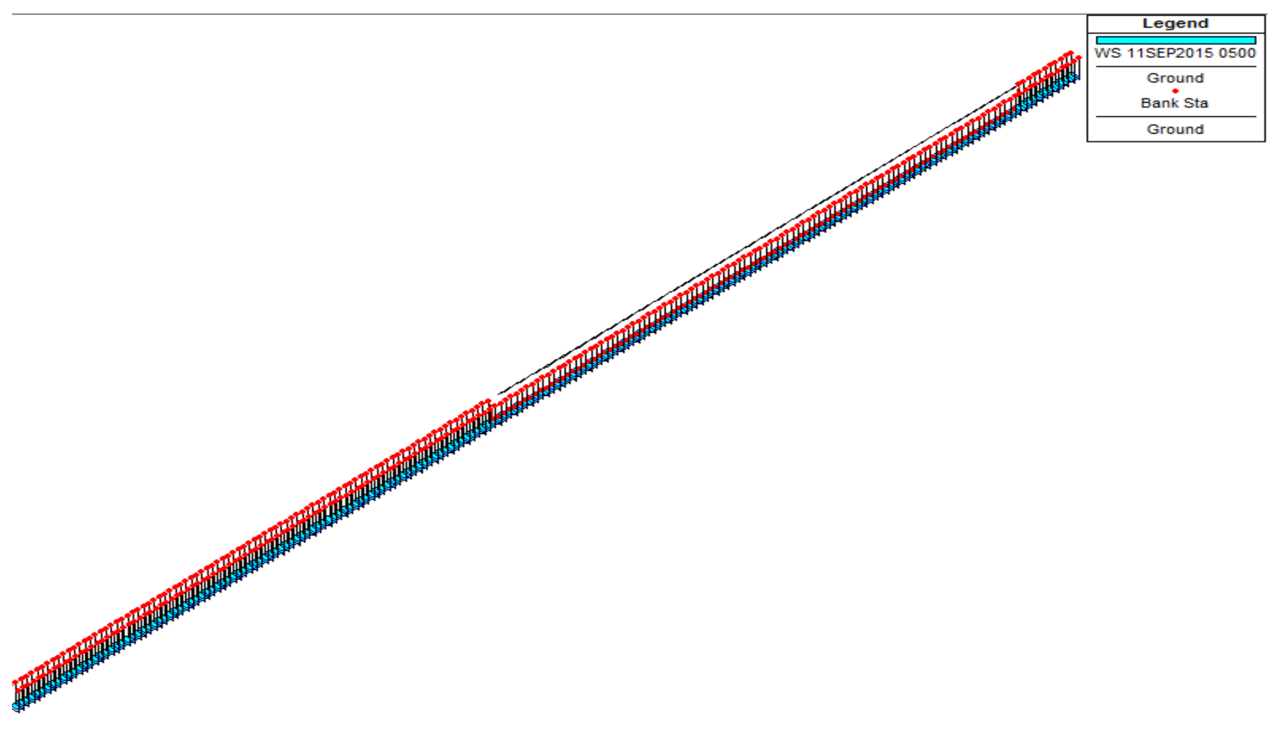


Figure 4.36 Levee Model Using *FLO-2D*

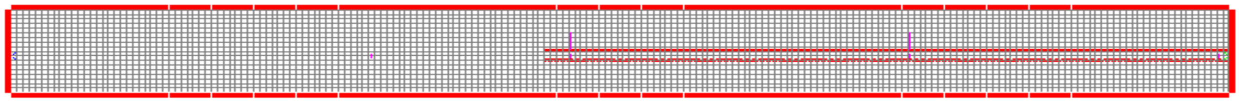


Table 4.15 Comparison of *FLO-2D* Levee with *HEC-RAS*									
***HEC-RAS* River Station**	***FLO-2D* Grid Number**	**Depth**		**Velocity**		**Area**		**Calculated Discharge**	
		RAS	**2D**	**RAS**	**2D**	**RAS**	**2D**	**RAS**	**2D**
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**
2009	2009	1.586	1.473	6.244	6.785	22.423	20.831	140.0	140.8
1997	1997	1.586	1.474	6.244	6.786	22.423	20.845	140.0	141.5
1900	1900	1.586	1.480	6.244	6.805	22.423	20.930	140.0	142.4

4.2.4.3 Scenario 2: Levee Breach Model

The purpose of this test is to verify that hydraulic calculations for the *FLO-2D* levee break option are performed appropriately. The *FLO-2D* and *HEC-RAS* models described in the previous section were used for this test. The levee was breached in the *FLO-2D* model at grid element 1797 and in *HEC-RAS* model at cross section 1998.

The levee breach parameters set in *FLO-2D* are:

1. Levee failure direction = 2
2. Elevation to initiate failure = 87.5 feet
3. Duration that the levee will fail when the failure elevation is reached = 0.2 hrs
4. Final failure elevation = 0 (set to the floodplain elevation of 86.5 feet)
5. Maximum breach width = 10 feet
6. Rate of vertical levee failure = 54 ft/hr
7. Rate of horizontal levee failure = 21 ft/hr

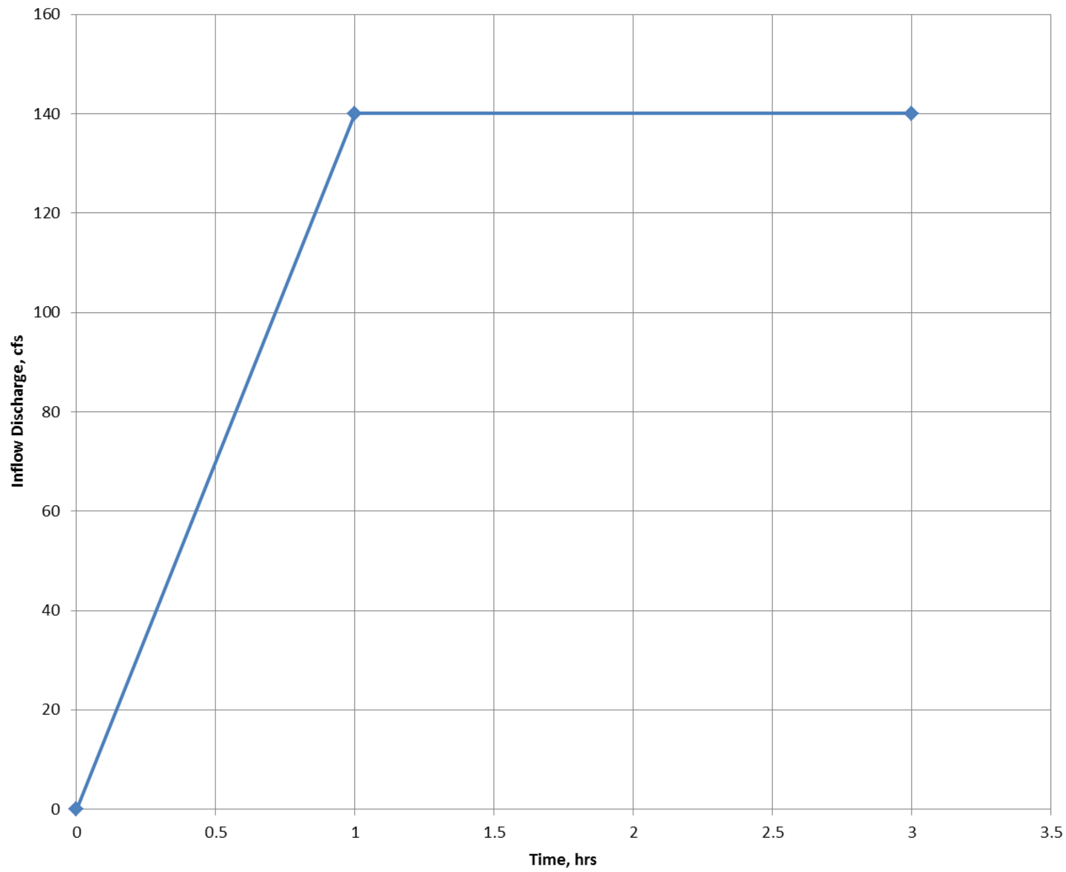
The initial breach width is hard coded to one foot. The maximum breach width is set to a width greater than one grid element side width (4.142 ft). *FLO-2D* automatically extends the breach into adjacent side elements (in this case, grid element sides 5 and 6) as necessary to meet the maximum breach width. For prescribed levee failures the side slopes are hard coded to vertical. The breach weir coefficient is also hard coded to 3.05.

The levee breach parameters set in *HEC-RAS* are:

1. Final Bottom Width = 10 feet
2. Final Bottom Elevation = 86.5 feet
3. Left Side Slope = 0
4. Right Side Slope = 0
5. Breach Weir Coefficient = 3.05
6. Full Formation Time = 0.8 hrs
7. Failure Mode = Piping
8. Piping Coefficient = 0.6
9. Initial Piping Elevation 86.5 feet
10. Trigger Failure at a water surface elevation of 87.5 feet

The inflow hydrograph for both models is shown on Figure 4.37. *HEC-RAS* requires constant flow in the model; therefore, the initial flow rate was set to 1 cfs. *FLO-2D* does not require a constant flow so the initial flow rate is set to zero. For *HEC-RAS* the model is set to discharge the flow over the breach out of the system. Outflow elements were placed around the downstream side of the breach in *FLO-2D* to mimic this condition. Since there is a potential for supercritical flows during the levee breach the mixed flow regime was selected in the *HEC-RAS* model. In the *FLO-2D* model, the limiting Froude number was set to zero (no Froude number limitations).

Figure 4.37 Inflow Hydrograph for Levee Breach Model



The breach peak discharge computed using *FLO-2D* is 40.8 cfs and for *HEC-RAS* is 44.41 cfs. The breach hydrographs are shown on Figure 4.38. Both programs use the broad-crest weir equation to calculate the flow over the breach.

$$Q = C_w \, L \, H^{\frac{3}{2}}$$

where:

- Q = discharge over the weir, cfs
- C_w = weir discharge coefficient
- L = weir length, feet
- H = depth of water over the weir, feet

The differences in peak discharge are due to the differences in calculating the depth of water over the weir. *FLO-2D* calculates a depth for each grid element side based on the adjoining grid cell elevations and these elevations are shown on Figure 4.39. These depths were obtained from the LEVEE.OUT file. *HEC-RAS* calculates depth based on water surface elevation of the cross sections just upstream, through and just downstream of the breach. The average and maximum weir depth used by *HEC-RAS* is shown on Figure 4.39. These values were obtained from Profile Output Table – Lateral Structures.

The *FLO-2D* levee breach method performs calculations in an acceptable manner, and meets minimum FCDMC requirements.

Figure 4.38 Breach Hydrograph for Levee Breach Models

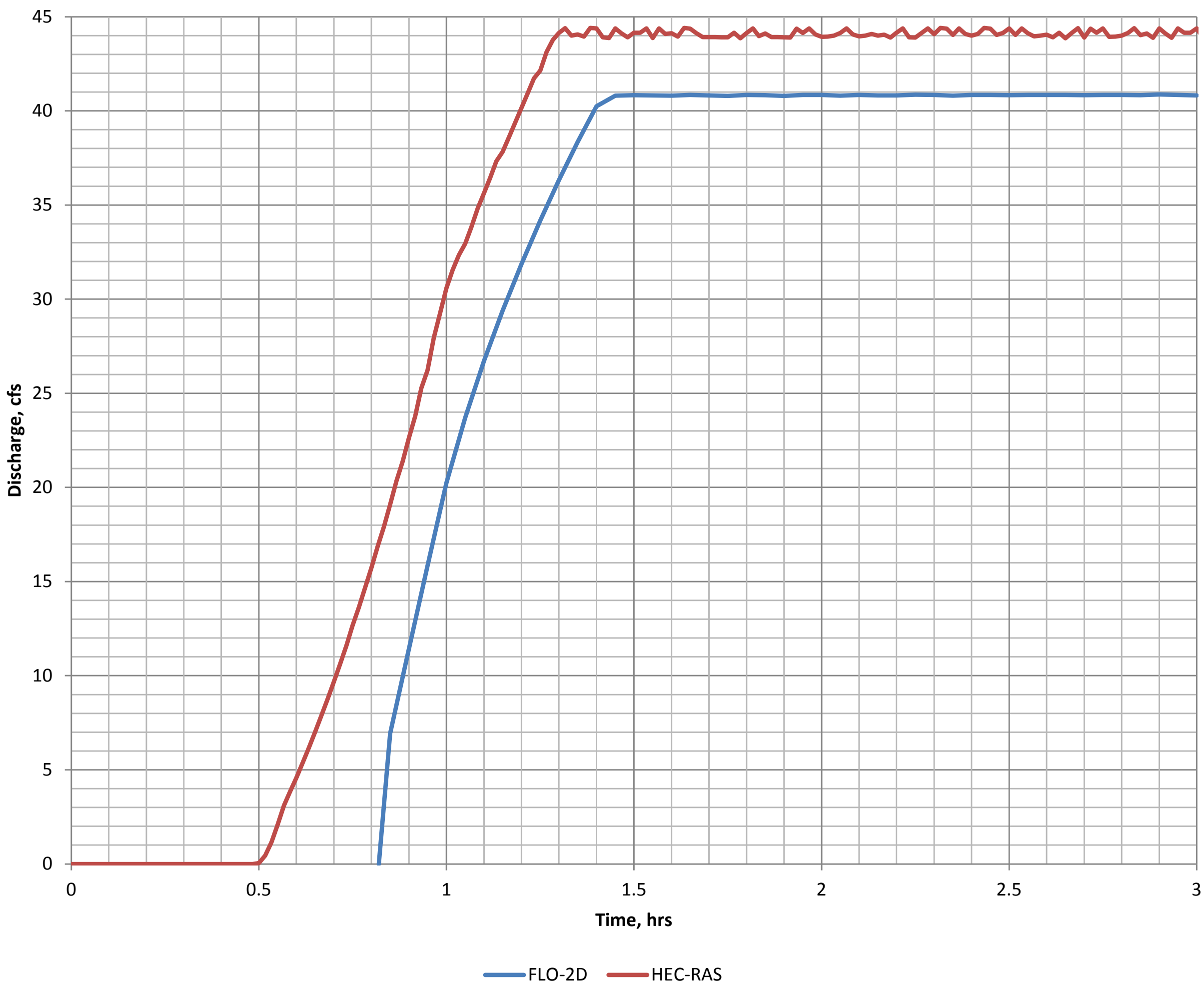
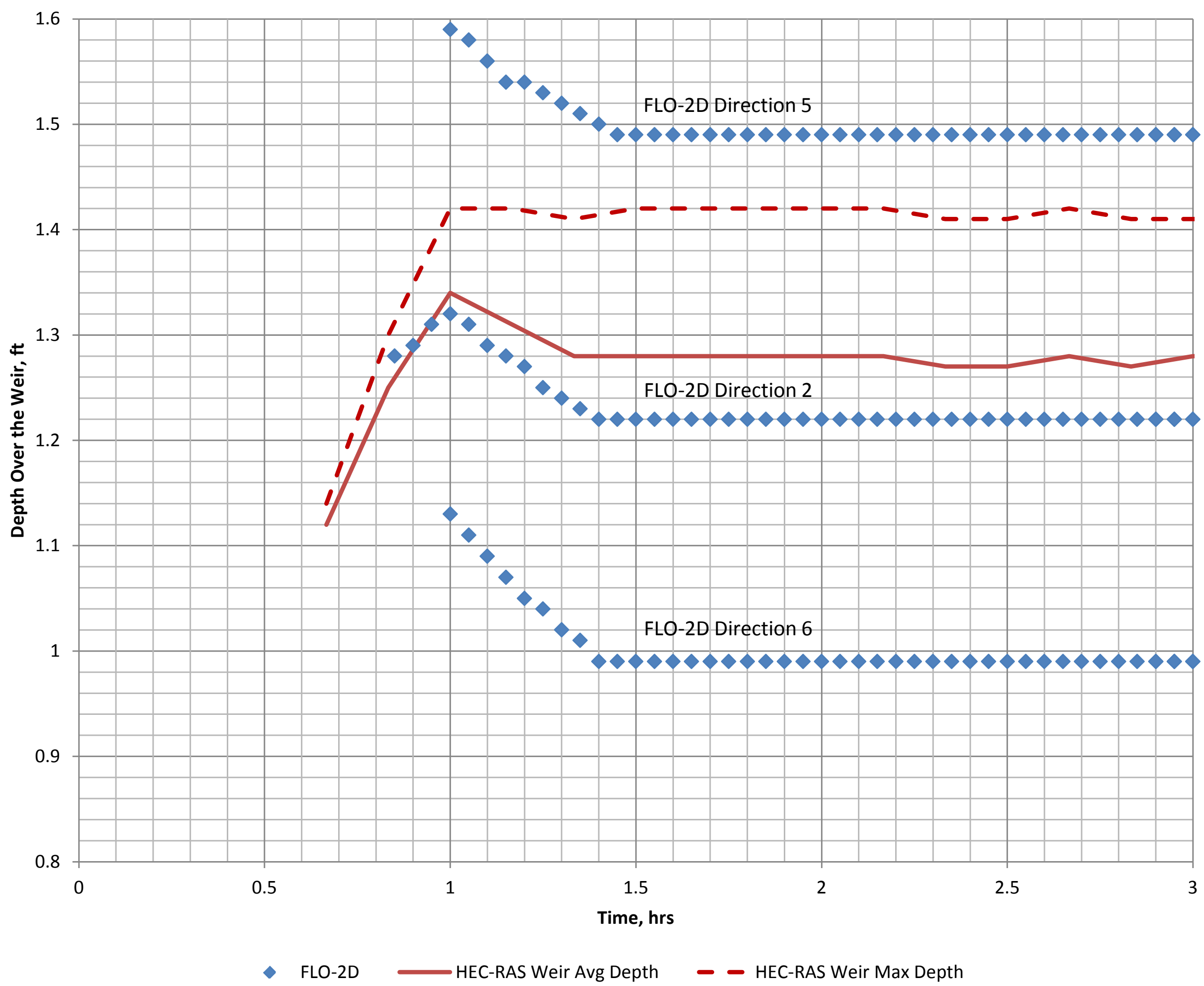


Figure 4.39 Depth Over the Weir for Levee Breach Models



4.2.5 Hydraulic Structures

4.2.5.1 General

The purpose of this section is to verify that hydraulic calculations for modeling hydraulic structures are done appropriately by comparing the *FLO-2D* results with results from the *HEC-RAS* computer program. The test cases include hydraulic structures that connect floodplain grids, *1D* channel elements, floodplain grids to *1D* channel elements, and *1D* channel elements to floodplain grids

The purpose of these tests is to verify that hydraulic structure calculations are performed appropriately for inlet control, outlet control, and the case where a culvert switches between inlet control and outlet control as flow increases or decreases. A *FLO-2D* model was built using 1,400 uniform square 20-foot grid elements, with a length of 100 grids and a width of 14 grids. For the floodplain grid test scenarios, the flume was modeled by insetting a rectangular channel simulated with a line of single grid elements forming the base, 10 feet below the grade of the adjacent elements as shown on Figure 4.40. For the *1D* channel *FLO-2D* test models, the prismatic *1D* channel option was used.

The benchmark used for verification of the *FLO-2D* model results was a direct comparison with *HEC-RAS* model results. The results of the unsteady *FLO-2D* test models using both the rating table and the general culvert equations options were compared with the *HEC-RAS* unsteady model results. The four benchmark comparisons described at the end of this section were done for all scenarios considered.

The *HEC-RAS* models were based on a uniform rectangular channel similar to the example shown on Figure 4.41 using an average cross section spacing of 100 feet. The test models consist of a rectangular flume with the following characteristics:

1. *HEC-RAS* Channel Base Width: 8.284 feet
2. *FLO-2D* Floodplain Grids Base Width: 20.00 feet
3. *FLO-2D 1D* Channels Base Width: 8.284
4. Depth: 10 feet
5. Outlet Control Slope: 0.0020 ft/ft
6. Inlet Control Slope: 0.0400 ft/ft
7. Inlet/Outlet Control Slope: 0.0045 ft/ft
8. *n*-value: Depth-variable *n* in both *FLO-2D* and *HEC-RAS*, with *n* = 0.05 for flow depths of 3 feet and greater for the floodplain grids, a bank-full depth of 10 feet for the *1D* channel models, and a shallow *n* of 0.20. The *HEC-RAS* depth-variable *n* option was implemented to mimic the *FLO-2D* approach.
9. Length: 2,000 feet
10. Culvert diameter: 3 feet
11. Box culvert dimensions: 3 feet x 3 feet
12. Culvert length: 120 feet for all models except a long culvert inlet control test case that used a culvert length of 1,400 feet.
13. Culvert material: Circular reinforced concrete pipe, *n* = 0.013
14. Culvert inlet: Square edge entrance with headwall, K_e = 0.5
15. Culvert slope: Same as upstream and downstream channels
16. The flow regime is sub-critical.

The following *HEC-RAS* plans were created for each model:

1. Steady State: A steady-state plan was created for development of culvert rating tables for input to *FLO-2D*. The rating table flow rates modeled were 0.01, 1, 5, 10, 20, 30, 40, 50, 60, 70, and 80 cfs.

2. Unsteady Varying Flow Hydrograph: An unsteady model using a rising and receding limb hydrograph. The peak discharge used varied depending on the model in order to test the response to different flow rates and head conditions. The peak discharges used were:
   A. Inlet control short culverts: 40 cfs
   B. Inlet control long culverts: 80 cfs
   C. Outlet control: 60 cfs
   D. Inlet/Outlet control: 60 cfs
3. Unsteady Uniform Flow Hydrograph: An unsteady model using a rising limb hydrograph that levels out and maintains the peak discharge for several hours. The peak flow rates were the same as used for the Unsteady Varying Flow Hydrograph models.

The results from the *HEC-RAS* steady state models were used to prepare hydraulic structure rating tables for use in the *FLO-2D* HYSTRUC.DAT input data file. The *FLO-2D* General Culvert Equations approach was also tested. The unsteady flow *HEC-RAS* models were used for direct comparison with the equivalent *FLO-2D* model results.

The *FLO-2D* model capabilities tested include:

1. Hydraulic structure rating table method for inlet control, outlet control and inlet/outlet control short culvert conditions.
2. Hydraulic structure General Culvert Equations method for inlet control, outlet control, and inlet/outlet control short culvert conditions for circular pipes.
3. Hydraulic Structure General Culvert Equations method for long culvert inlet control conditions for circular pipes.
4. Shallow *n* and depth-variable Manning's *n*-value option. The *FLO-2D* depth-variable *n*-value method was compared with *HEC-RAS* in all of the hydraulic structure test models. An example *HEC-RAS* depth-variable *n*-value table, designed to mimic the *FLO-2D* approach, is shown in Figure 4.42. The *FLO-2D* shallow *n* parameter was set to 0.20, and all grid and *1D* channel *n*-values for flow depths 3 feet and greater were set to 0.050.

The hydraulic structure General Culvert Equations method using box culverts was tested using the exact same scenarios as for circular pipes. Some small issues were noted and are being addressed with the software developer. As a result, it is recommended that the rating table method be applied when modeling box culverts until the discrepancies are addressed. The box culvert model results are not documented in this report but the test models and summary spreadsheets are available upon request.

The *HEC-RAS* models, and the *FLO-2D 1D* channel models, were created with a base width of 8.284 feet. This setting is intentional in order to accurately compare the results of the two models with the *FLO-2D* floodplain grid-only models that simulate the channel with a 20 foot wide grid element. Refer to Section 4.2.2.2 for a description of why the *1D* model base widths must be less than the *FLO-2D* grid width in order to compare hydraulic results.

Figure 4.40 *FLO-2D* Flume Model Surface Using Floodplain Grids

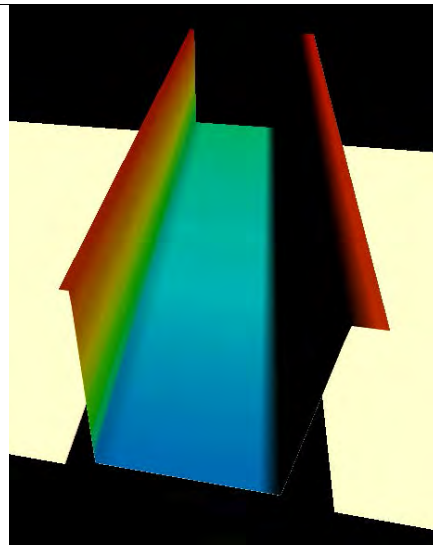


Figure 4.41 Flume Model Using *HEC-RAS*

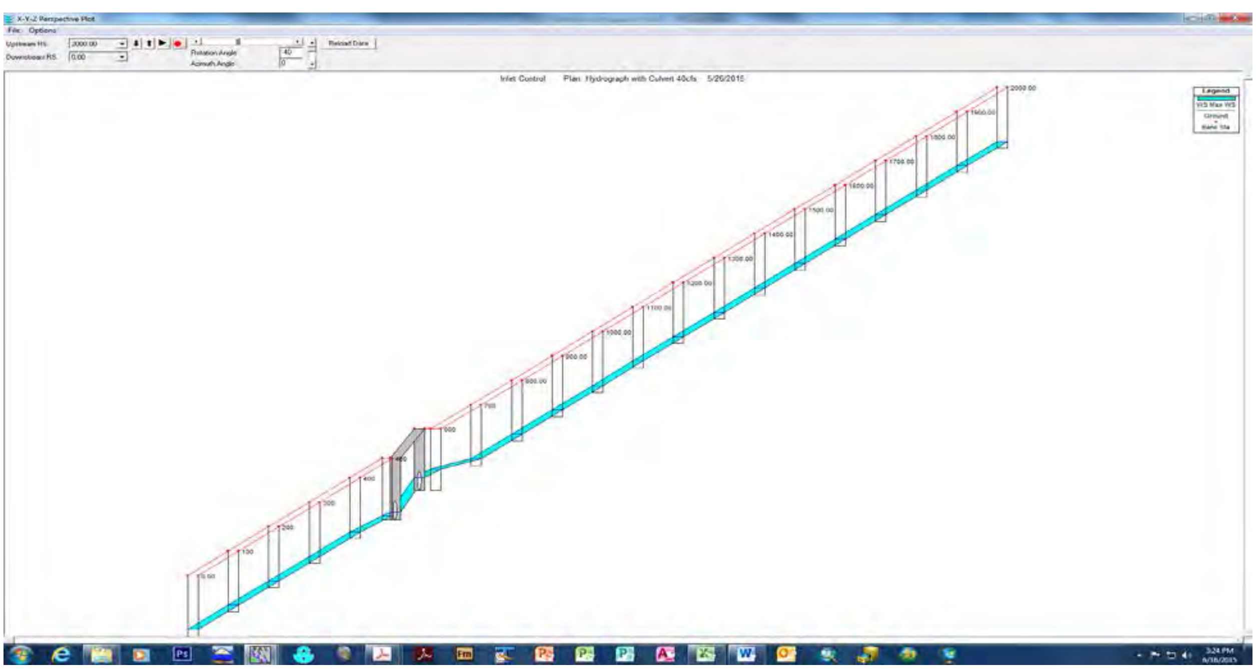
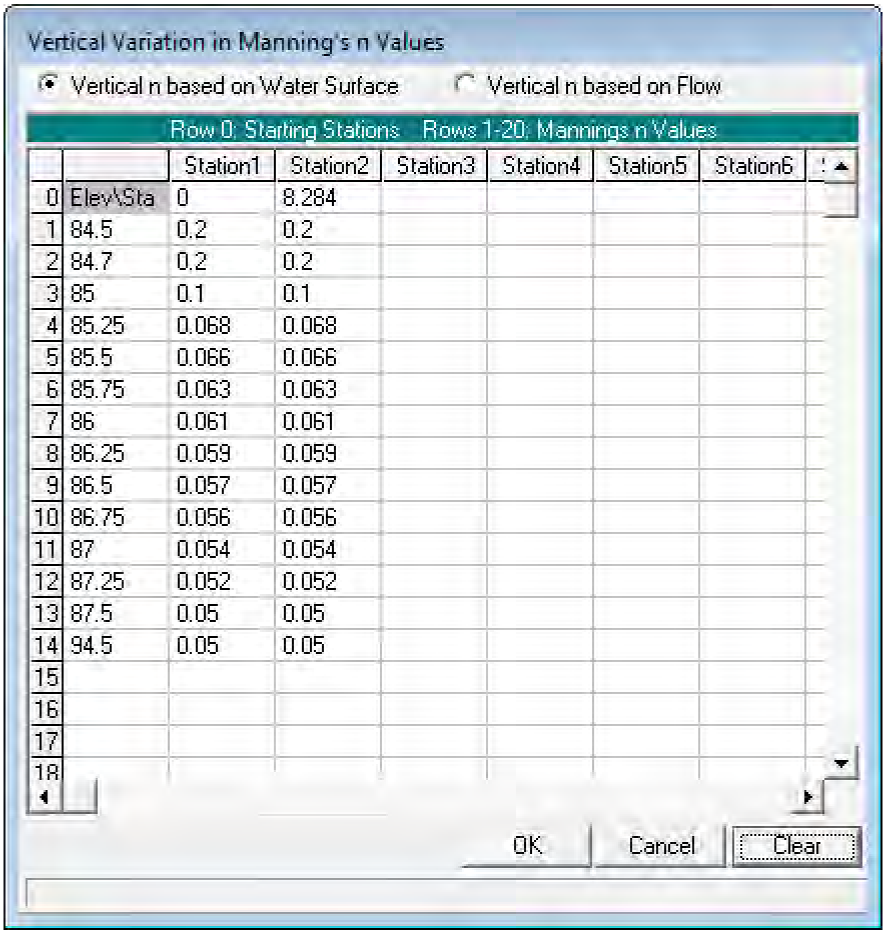


Figure 4.42 Example *HEC-RAS* Depth-Variable Manning's *n*-Values Table



Vertical Variation in Manning's n Values

Vertical n based on Water Surface   Vertical n based on Flow

Row 0: Starting Stations   Rows 1-20: Mannings n Values

		Station1	Station2	Station3	Station4	Station5	Station6
0	Elev\Sta	0	8.284				
1	84.5	0.2	0.2				
2	84.7	0.2	0.2				
3	85	0.1	0.1				
4	85.25	0.068	0.068				
5	85.5	0.066	0.066				
6	85.75	0.063	0.063				
7	86	0.061	0.061				
8	86.25	0.059	0.059				
9	86.5	0.057	0.057				
10	86.75	0.056	0.056				
11	87	0.054	0.054				
12	87.25	0.052	0.052				
13	87.5	0.05	0.05				
14	94.5	0.05	0.05				
15							
16							
17							
18							

OK   Cancel   Clear

For outlet control models, the Limiting *Froude number* is set to 0.2 to match the *HEC-RAS* model results, which have a *Froude number* of 0.2 or less. The *FLO-2D* floodplain and *1D* channel Courant parameters were set to 0.2 for all test models. Refer to Section 4.2.5.19 for a discussion of possible effects on results when using other values of Courant. The *HEC-RAS* unsteady numerical control parameters were also adjusted as needed for a stable model. The computation interval was set to 20 seconds, time slicing was enabled with a minimum time step of 0.001 hours, Theta was set to 0.6, and the maximum number of iterations and time slices was increased from 20 to 40. The HTAB parameter tables were set to the channel invert elevation.

Verification models were prepared for the following scenarios (OC = Outlet Control, IC = Inlet Control):

1. Scenario 1: Rating Table Floodplain to Floodplain (OC).
2. Scenario 2: General Equations Floodplain to Floodplain (OC).
3. Scenario 3: Rating Table Floodplain to Floodplain (IC).

4. Scenario 4: General Equations Floodplain to Floodplain (IC)
5. Scenario 5: Rating Table Floodplain to Floodplain (IC and OC)
6. Scenario 6: General Equations Floodplain to Floodplain (IC and OC)
7. Scenario 7: Rating Table Channel to Channel (OC).
8. Scenario 8: General Equations Channel to Channel (OC).
9. Scenario 9: Rating Table Channel to Channel (IC and OC)
10. Scenario 10: General Equations Channel to Channel (IC and OC).
11. Scenario 11: General Equations Channel to Channel (IC and OC) – with backwater
12. Scenario 12: Rating Table Floodplain to Channel (OC).
13. Scenario 13: General Equations Floodplain to Channel (OC).
14. Scenario 14: Rating Table Channel to Floodplain (OC).
15. Scenario 15: General Equations Channel to Floodplain (OC).
16. Scenario 16: Rating Table Long Culvert Channel to Floodplain (IC).
17. Scenario 17: General Equations Long Culvert Channel to Floodplain (IC).

The benchmark comparisons were made as follows for each scenario:

1. Culvert inlet *WSEL* versus Discharge (varying discharge unsteady flow model only). The data used is for all model time steps from both *FLO-2D* and unsteady *HEC-RAS*, and compared to the input rating table generated using steady state *HEC-RAS*. The *FLO-2D* sources for WSEL are the HYCROSS.OUT (culvert inlet floodplain grid 631 or 686) and HYCHAN.OUT (*1D* channel grids 631 or 686) files. HYCROSS.OUT is used as the WSEL source for floodplain grid test cases, and HYCHAN.OUT for 1D channel test cases. The *FLO-2D* source for culvert discharge is the HYDROSTRUCT.OUT file. The *HEC-RAS* source is the Rating Curve option.
2. Culvert inlet Discharge over Time (both varying and uniform discharge unsteady flow models). Discharge entering the culvert inlet over time compared with model inflow discharge over time for both *FLO-2D* and unsteadies *HEC-RAS*. The *FLO-2D* source is the HYDROSTRUCT.OUT file. The *HEC-RAS* source is the Stage and Flow Hydrographs option.
3. Maximum *WSEL* Profiles (both varying and uniform discharge unsteady flow models). Maximum *FLO-2D WSEL* profile compared with unsteady *HEC-RAS* maximum *WSEL* for the entire model length, with the thalweg ground profile also shown. The *FLO-2D* source is the MAXQHYD.OUT or MAXWSELEV.OUT files for floodplain grids and the CHANWS.OUT file for *1D* channel grids. The *HEC-RAS* source is the View Profiles option.
4. Culvert inlet *WSEL* over Time (uniform discharge unsteady flow model only). The *FLO-2D* sources are the HYCROSS.OUT (culvert inlet floodplain grid 631 or 686) and HYCHAN.OUT (*1D* channel grids 631 or 686) files. The *HEC-RAS* source is the Stage and Flow Hydrographs option.

All the charts included in the hydraulic structure test results use blue lines for *HEC-RAS* and red lines for *FLO-2D*. The details and results for each scenario are discussed in the following sections.

4.2.5.2 Scenario 1: Rating Table Floodplain to Floodplain (OC)

Scenario

Test of the *FLO-2D* rating table approach for a culvert connecting two floodplain grids, operating under outlet control.

Results and Discussion

This scenario tests a culvert connecting two floodplain grids with the topography configured to force an outlet control condition for all test flow rates. The rating table developed using *HEC-RAS*, shown as a black line on Figure 4.43, was used to control the *FLO-2D* hydraulic structure routine. The *FLO-2D* INOUTCONT variable was set to 0 to force no adjustment due to tailwater. The rating table from *HEC-RAS* is based on outlet control, which is affected by tailwater. *FLO-2D* applies this rating table by using the flow depth at the inlet to obtain the culvert discharge. In a real-world scenario, if the tailwater elevation computed by *FLO-2D* is different than was assumed when developing the rating table, the *FLO-2D* results may not be theoretically correct and only an approximation. This should be considered when using this option.

A comparison of *WSEL* and discharge results at the culvert inlet and outlet is shown in Table 4.16. The minor differences between the *FLO-2D* and unsteady *HEC-RAS* results are expected and acceptable for *FCDMC* purposes.

Benchmark 1 Results. The unsteady output depth-discharge results (with depth represented as *WSEL*) from *FLO-2D* (red dotted line) at the culvert inlet are compared with the *HEC-RAS* results (blue dashed line) on Figure 4.43 for the varying flow hydrograph condition. The flow depths on the hydraulic structure inlet grid were extracted in 0.05 hour time steps from the HYCROSS.OUT file and converted to *WSEL*. The peak discharges conveyed by the culvert for each 0.05 hour time step were extracted from the HYDROSTRUCT.OUT file. The *FLO-2D* results almost exactly match the *HEC-RAS* results.

Benchmark 2 Results. The model inflow and culvert inflow hydrographs are plotted on Figure 4.44 and Figure 4.45 for the varying flow and uniform hydrograph conditions, respectively. The *FLO-2D* inflow hydrograph (dotted red line) and the *HEC-RAS* inflow hydrograph (dashed blue line) are identical, which is a check that the input data files match. The *FLO-2D* culvert hydrograph (solid red line) and the *HEC-RAS* culvert hydrograph (solid blue line) are nearly identical. There are minor discrepancies that are to be expected when comparing results from two different models, but the differences are not significant.

Table 4.16 Scenario 1 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	80.25	79.04	38.90	1.25
HEC-RAS Varying	80.23	79.12	38.42	
FLO-2D Uniform	80.33	79.10	40.00	0.00
HEC-RAS Uniform	80.36	79.18	40.00	

Figure 4.43 Scenario 1 Rating Table Floodplain to Floodplain (OC) Test Results

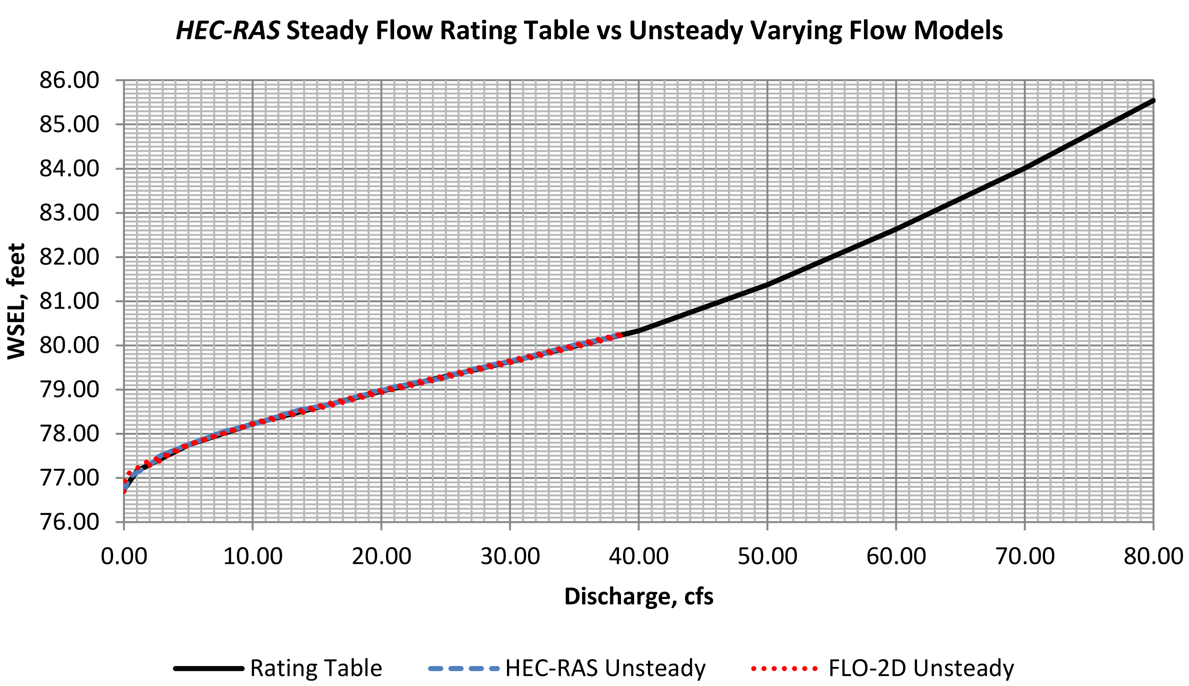


Figure 4.44 Scenario 1 Varying Flow Hydrograph Results

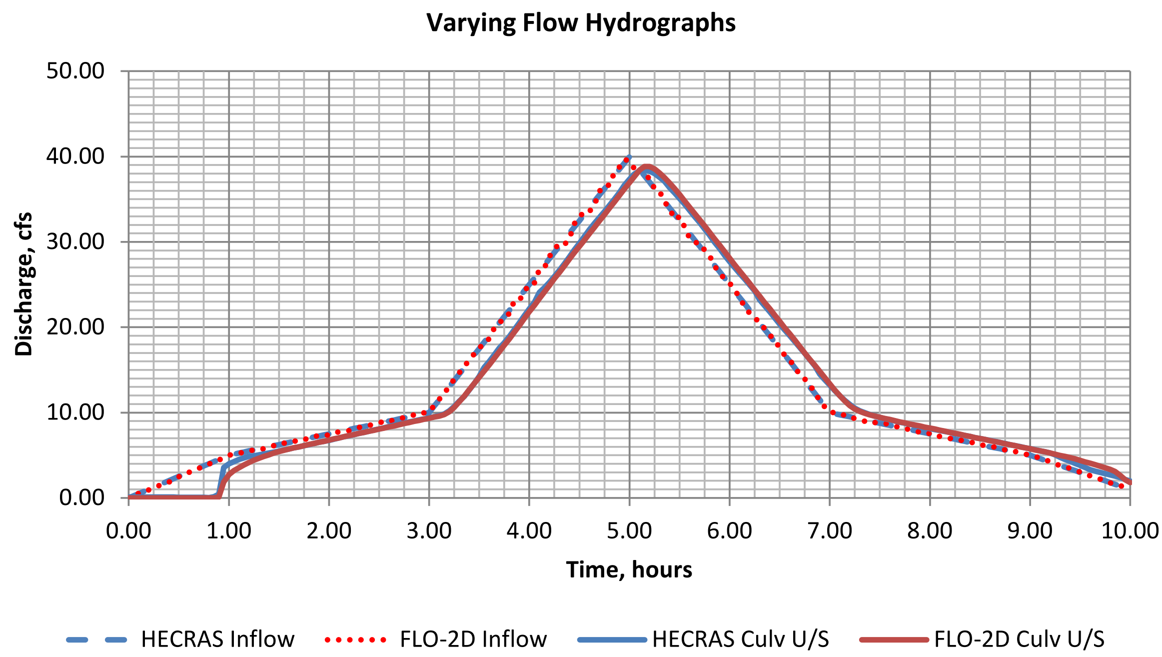
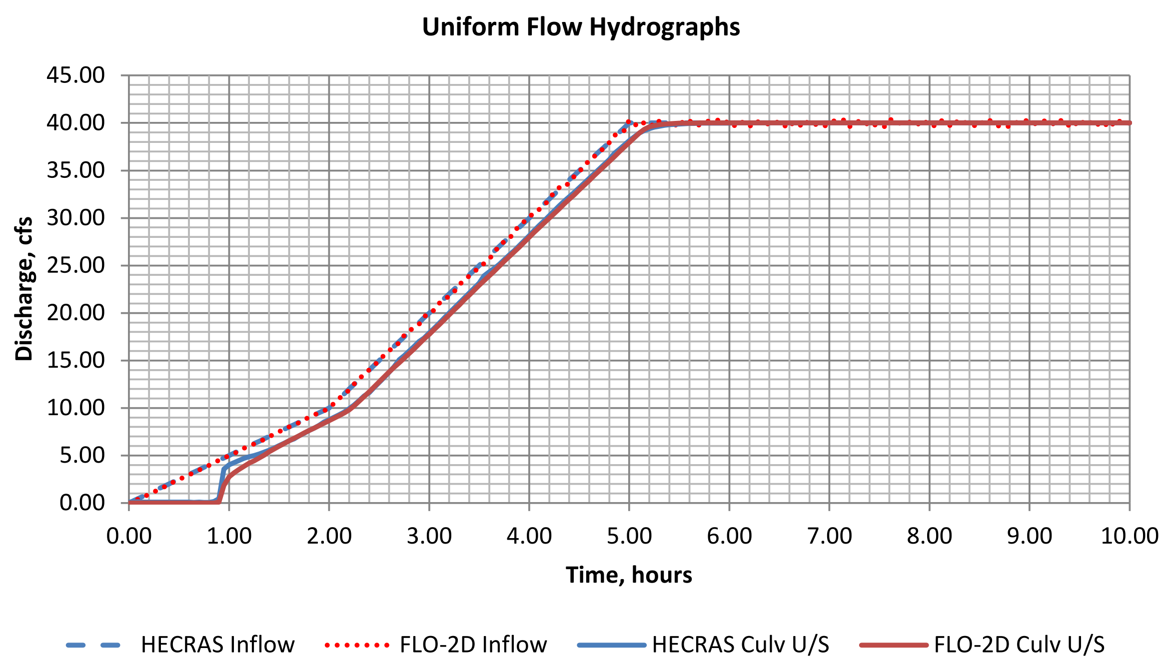


Figure 4.45 Scenario 1 Uniform Flow Hydrograph Results



A Root Mean Square Deviation (*RMSD*) analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.44 and Figure 4.45. The *RMSD* for the varying flow culvert inlet hydrograph was 0.33 cfs, which is a very minor deviation.

Benchmark 3 Results. A plot of maximum *WSEL* profile is shown on Figure 4.46 and Figure 4.47 for the varying flow and uniform flow hydrograph conditions, respectively. Note that *FLO-2D* does not report *WSEL* within the culvert so those values are zero, which results in the *FLO-2D WSEL* profile dropping to elevation 0.00 through the pipe. This is typical for all the hydraulic structure test scenarios. The *WSEL* immediately upstream and downstream of the culvert are a close match. Downstream of the culvert, *FLO-2D* has a profile approximately 0.1 feet lower than *HEC-RAS*. Upstream from the culvert, the *FLO-2D* profile is nearly an exact match with *HEC-RAS,* transitioning to be 0.1 feet lower at the upstream end.

Since the *FLO-2D* model for this case is based on floodplain grids, there is a difference in the conveyance calculation between *FLO-2D* and *HEC-RAS* that accounts for the *FLO-2D* profile being lower downstream of the culvert and areas not influenced by backwater effects. *FLO-2D* floodplain grids are intended to model shallow unconfined overland flow. Therefore, the wetted perimeter for *FLO-2D* does not include the two channel sides, only the base flow width. This results in a slightly higher velocity and a lower flow depth. For instance, from the *HEC-RAS* Unsteady Uniform Flow test model at Station 2+00 (Grid 611 from MAXQHYD.OUT) and at time 9.61 hours, consider the data and results in Table 4.17. The depth computed by both *FLO-2D* and *HEC-RAS* are listed, with the *HEC-RAS* depth being about 0.1 feet higher than *FLO-2D*.

The slope of the energy grade line reported by *FLO-2D* is 0.00205 ft/ft and by *HEC-RAS* is 0.00200 ft/ft. Therefore, at this location and time, flow can be considered uniform and backwater effects are minimal. *HEC-RAS* used an *n*-value of 0.052 and *FLO-2D* a value of 0.070. The flow depth is less than 3 feet so the depth variable *n*-value adjustment resulted in the slight increase over the 0.05 base for *HEC-RAS*. *FLO-2D* used a higher *n*-value resulting from the 0.2 limiting *Froude number* setting in combination with depth-variable n. The velocity computed by hand calculation (highlighted in yellow in Table 4.17) agrees with the values calculated by *FLO-2D* and *HEC-RAS*.

The difference in wetted perimeter and *n*-value are the reasons why *FLO-2D* computes a lower *WSEL* than *HEC-RAS* for this test case downstream of the culvert. A comparison of velocities over time between *FLO-2D* and *HEC-RAS* at various stations are shown on Figure 4.48. Note that the *FLO-2D* velocities are also greater than *HEC-RAS* for this scenario for the same reasons.

A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.46 and Figure 4.47. The *RMSD* for the varying maximum *WSEL* profile was 0.15 feet, which is a very minor deviation.

Benchmark 4 Results. A plot of *WSEL* over time at the culvert inlet is shown on Figure 4.49 from the uniform flow hydrograph condition. The *FLO-2D* results are almost an exact match with unsteady *HEC-RAS*.

The results of the Scenario 1 testing are acceptable for *FCDMC* purposes.

Figure 4.46 Scenario 1 Varying Flow *WSEL* Profile Results

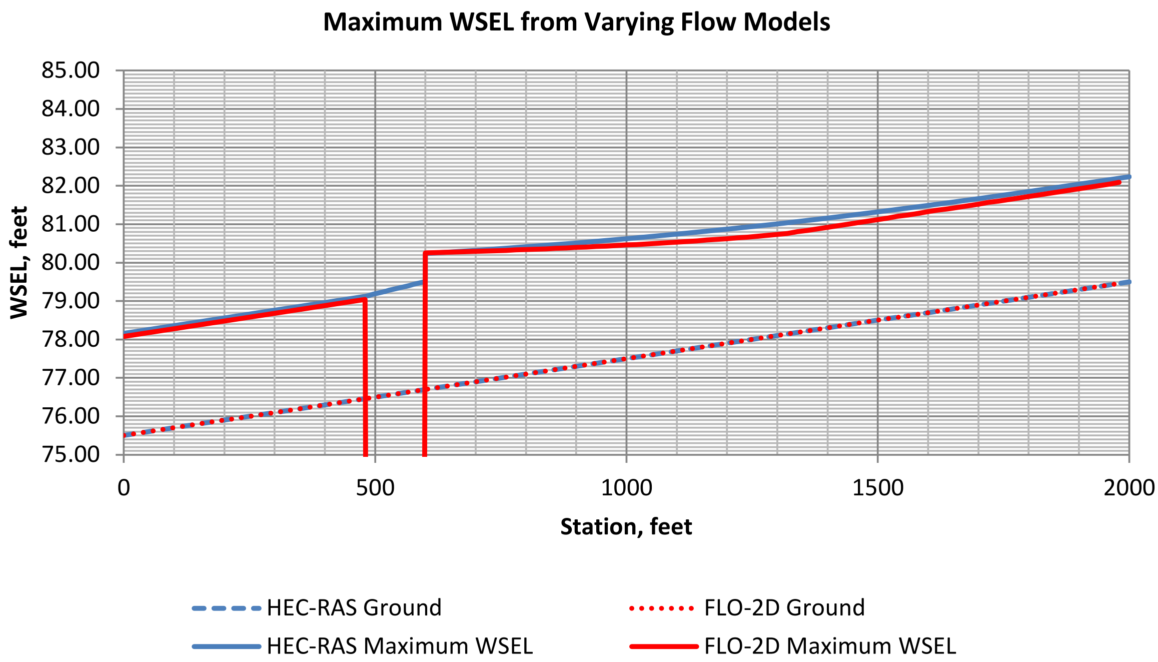


Figure 4.47 Scenario 1 Uniform Flow *WSEL* Profile Results

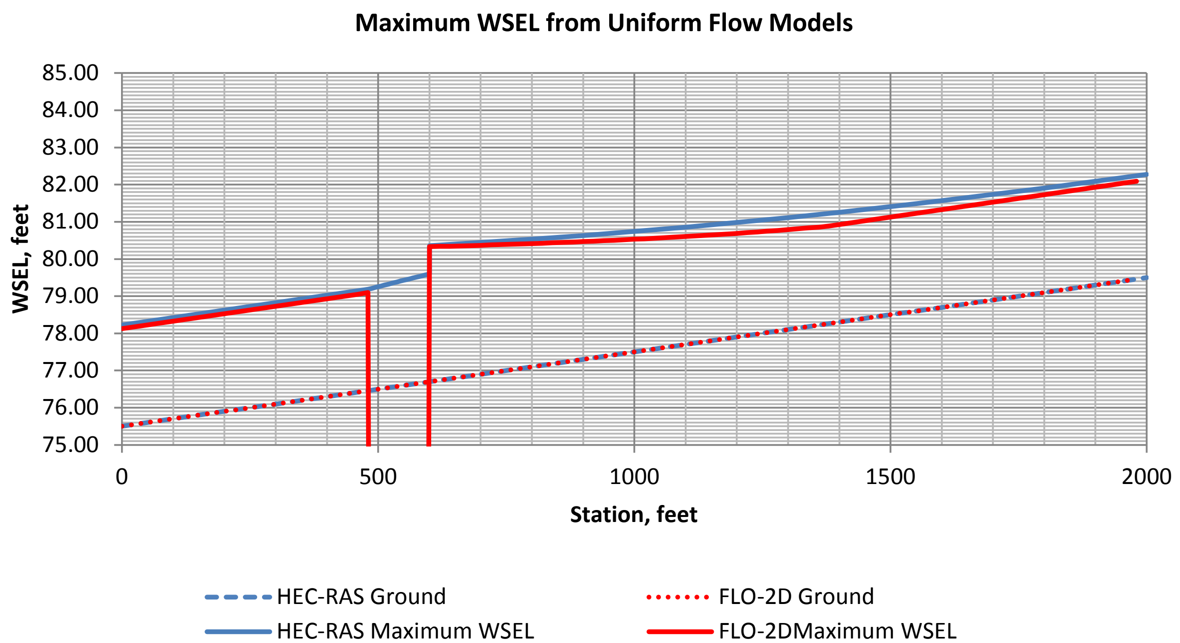


Table 4.17 Scenario 1 Comparison of Depth and Velocity at Station 2+00 (Grid 611)

***FLO-2D* at Time 9.61 hours**					***HEC-RAS***				
Depth, ft	**Area, sf**	**Wetted Perimeter, ft**	**n**	**Velocity, fps**	**Depth, ft**	**Area, sf**	**Wetted Perimeter, ft**	**n**	**Velocity, fps**
2.63	21.79	8.28	0.070[1]	1.81	2.72	22.56	13.72	0.052	1.78
Computed by *FLO-2D*:				1.85	Computed by *HEC-RAS*:				1.77
Discharge by *FLO-2D*:				40.19	Discharge:				39.93
1	A limiting *Froude number* of 0.20 is applied to match the *HEC-RAS Froude number*. This results in *n*-value adjustments that increase the *n*-value to 0.070, helping account for the difference in wetted perimeter								
	Computed using Manning's equation								

Figure 4.48 Scenario 1 Uniform Flow Velocity Results

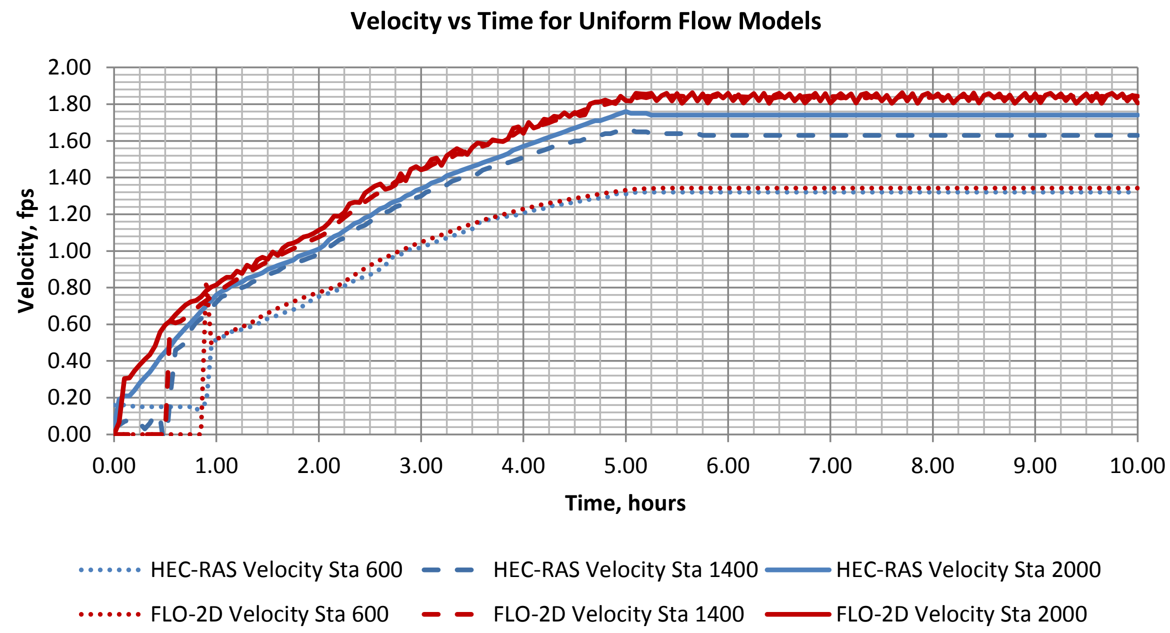
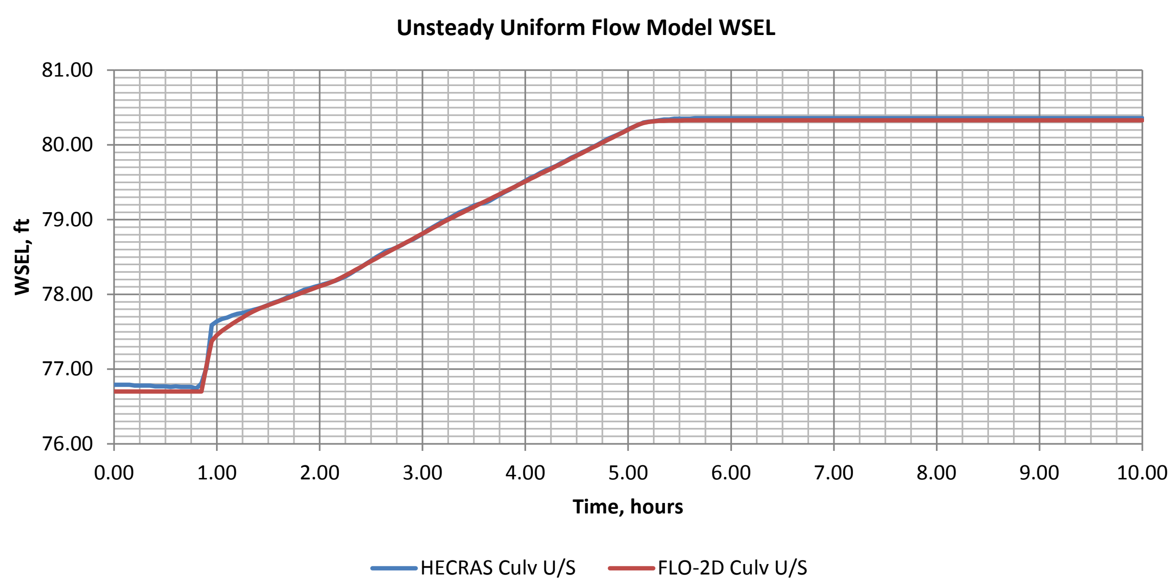


Figure 4.49 Scenario 1 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.3 Scenario 2: General Culvert Equations Floodplain to Floodplain (OC)

Scenario

Test of the *FLO-2D* general culvert equations approach for a culvert connecting two floodplain grids, operating under outlet control.

Results and Discussion

Scenario 2 is the same as Scenario 1 except that the general culvert equations option described in Section 2.4.8.1 was applied instead of a rating table. For this scenario the INOUTCONT variable was set to 2. The culvert was assigned an *n*-value of 0.013 and a K_e of 0.5 for a square edge with headwall inlet type. The key difference between the two models is tailwater computation. The general culvert equations method checks for both inlet and outlet control using the upstream and downstream heads, while the rating table approach only evaluates using the upstream head and a rating table that may or may not include outlet control effects. Refer to Section 2.4.8.1 for more description. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.18. The *FLO-2D WSEL* at the inlet for the uniform flow hydrograph condition is 0.17 higher than *HEC-RAS*. Otherwise the results are comparable with *HEC-RAS*.

The results from Benchmark 1 are shown on Figure 4.50. The general culvert equation-based results are slightly unstable between 15 and 25 cfs but compare very well with the steady state *HEC-RAS* rating table and unsteady *HEC-RAS*.

The results from Benchmark 2 are shown on Figure 4.51 and Figure 4.52. The *FLO-2D* results compare very well with unsteady *HEC-RAS*. A Root Mean Square Deviation (*RMSD*) analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.51 and Figure 4.52. The *RMSD* for the varying flow culvert inlet hydrograph was 0.37 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.53 and Figure 4.54. The *WSEL* profile downstream of the culvert for the varying and uniform flow models is slightly lower than *HEC-RAS*, similar to the results for Scenario 1. The varying flow model *WSEL* profile upstream of the culvert also matches Scenario 1 and is a good check with *HEC-RAS*. The uniform flow model *WSEL* profile upstream of the culvert varies from 0.0 to 0.2 feet higher than *HEC-RAS*.

A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles (Courant = 0.2) as a way to quantify the significance of the differences noted on Figure 4.53 and Figure 4.54. The *RMSD* for the varying maximum *WSEL* profile was 0.15 feet, which is a very minor deviation.

The results from Benchmark 4 are shown on Figure 4.55. The *FLO-2D WSEL* varies from 0.0 to 0.25 feet higher than *HEC-RAS* and there is numeric instability after the peak uniform flow rate is reached, but the results are acceptable for *FCDMC* purposes.

The minor differences between the *FLO-2D* and unsteady *HEC-RAS* results from Scenario 2 are expected and acceptable for *FCDMC* purposes.

Table 4.18 Scenario 2 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	80.26	79.04	38.87	1.17
HEC-RAS Varying	80.23	79.12	38.42	
FLO-2D Uniform	80.51	79.09	40.03	0.08
HEC-RAS Uniform	80.36	79.18	40.00	

Figure 4.50 Scenario 2 Rating Table Floodplain to Floodplain (OC) Test Results

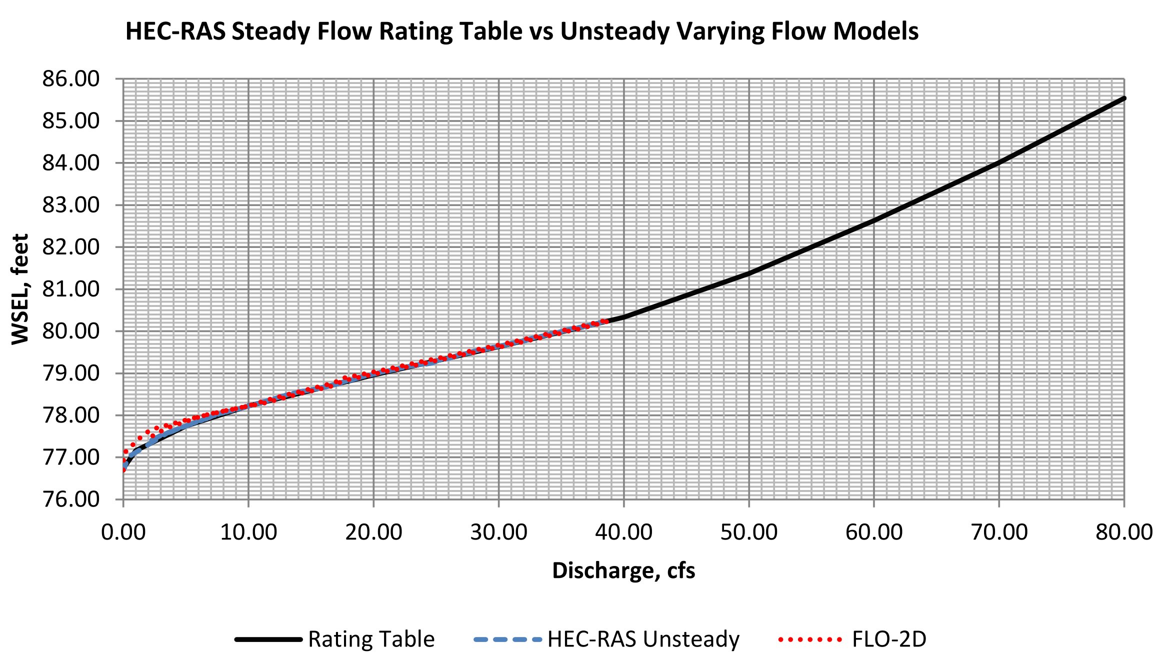


Figure 4.51 Scenario 2 Varying Flow Hydrograph Results

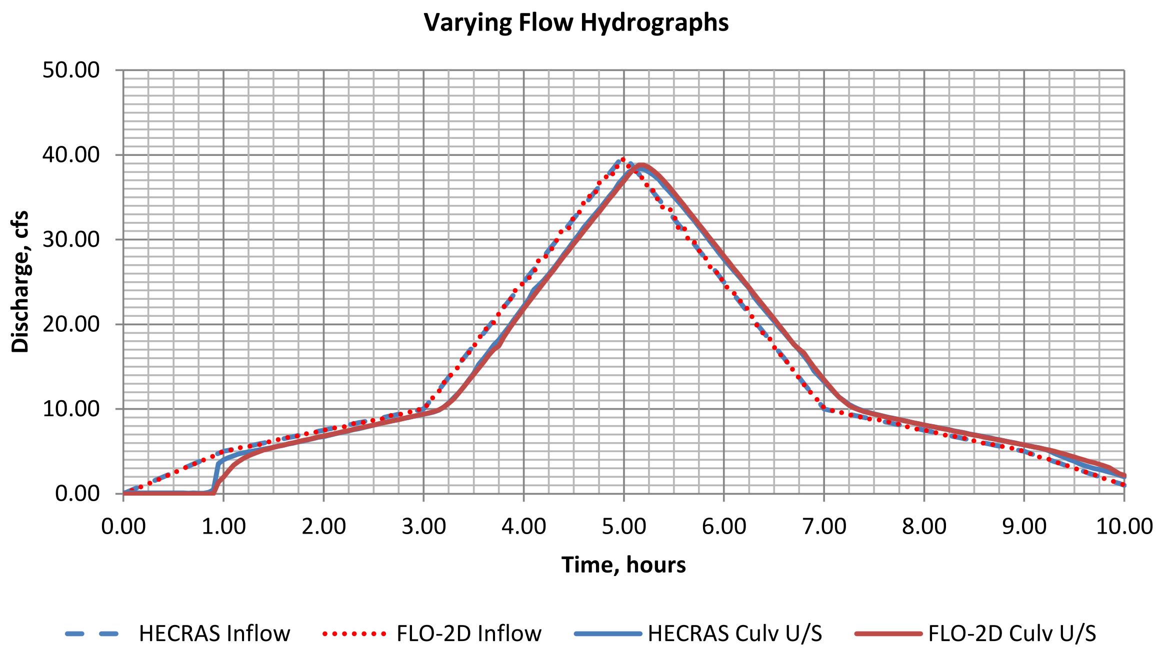


Figure 4.52 Scenario 2 Uniform Flow Hydrograph Results

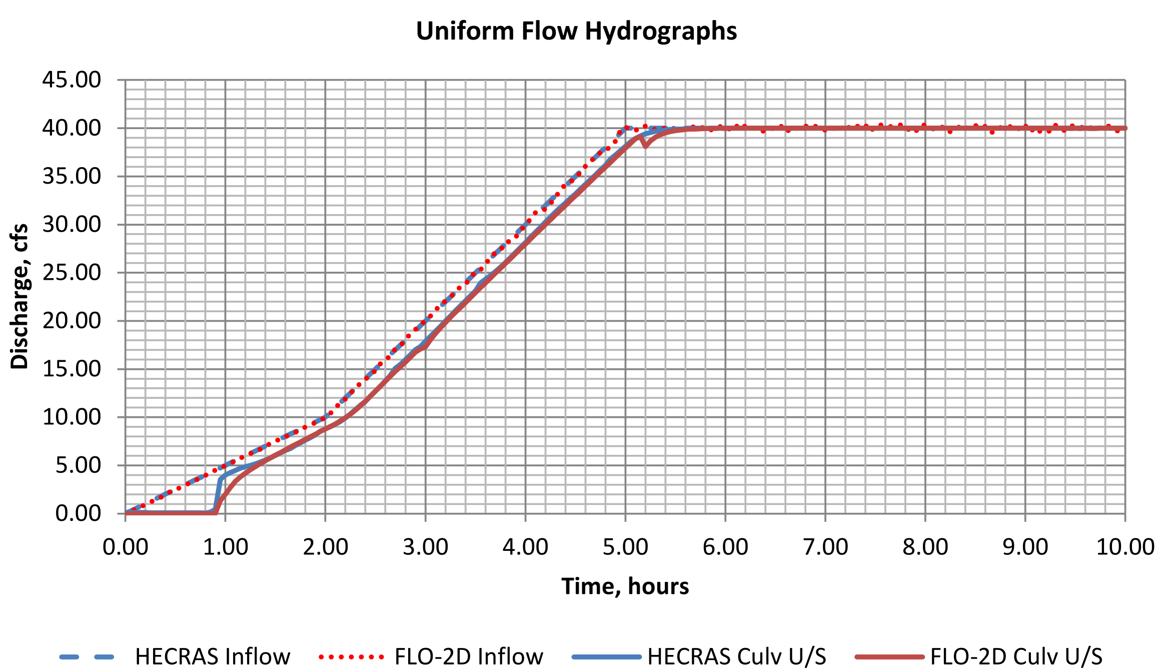


Figure 4.53 Scenario 2 Varying Flow *WSEL* Profile Results

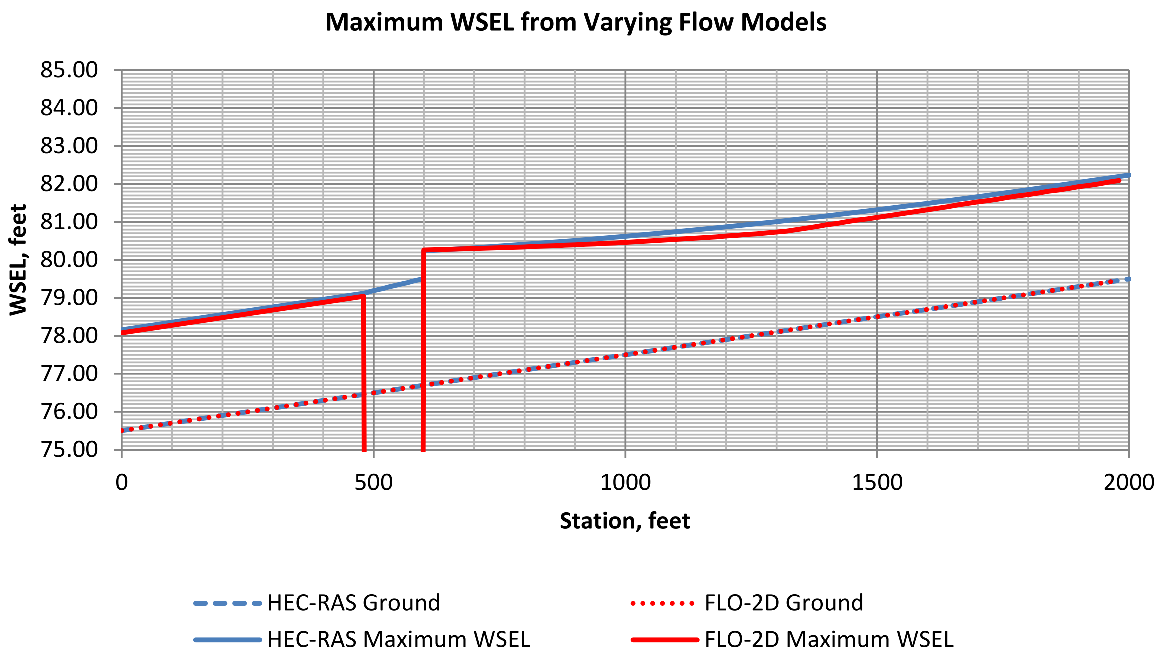


Figure 4.54 Scenario 2 Uniform Flow *WSEL* Profile Results

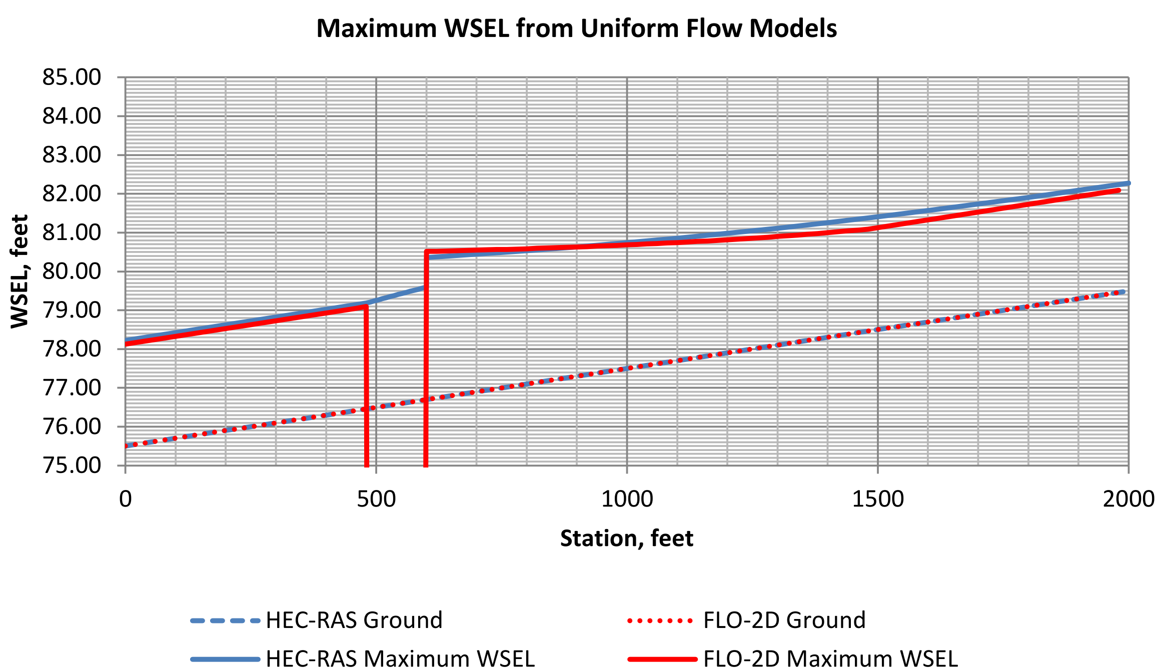
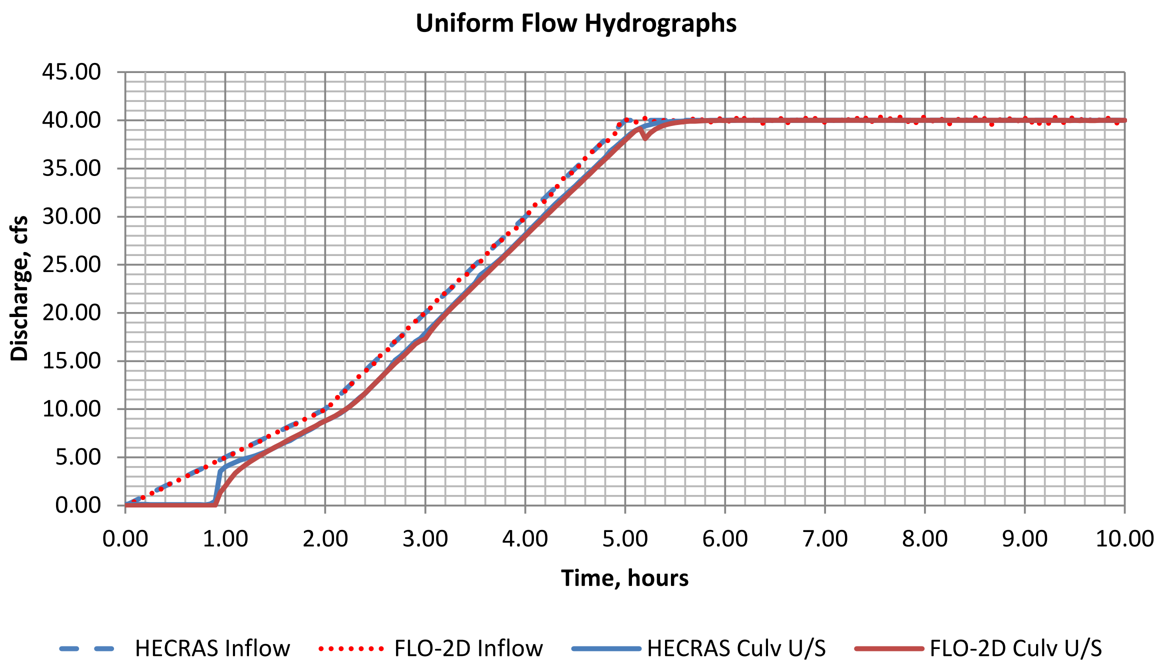


Figure 4.55 Scenario 2 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.4 Scenario 3: Rating Table Floodplain to Floodplain (IC)

Scenario

Test of the *FLO-2D* rating table approach for a culvert connecting two floodplain grids, operating under inlet control. The ground and culvert slopes were increased to 4.000% to force an inlet control condition.

Results and Discussion

Scenario 3 is the same as Scenario 1 except that the slope was increased to force an inlet control condition. The results are compared with the unsteady *HEC-RAS* inlet control model. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.19. The maximum *WSEL* at the culvert inlet is an exact match with *HEC-RAS*. The maximum *WSEL* at the culvert outlet is 0.5 feet higher than *HEC-RAS*. *HEC-RAS* is reporting velocity leaving the culvert and *FLO-2D* is reporting the normal depth velocity in the channel. Immediately downstream of the culvert, the results agree for the remainder of the downstream reach. Upstream of the culvert, the results agree.

The results from Benchmark 1 are shown on Figure 4.56. The *FLO-2D* results match the steady state *HEC-RAS* rating table and unsteady *HEC-RAS* except for a small amount of numerical instability between 8 and 13 cfs.

The results from Benchmark 2 are shown on Figure 4.57 and Figure 4.58. Although a little unstable between 8 and 13 cfs, the *FLO-2D* results compare very well with unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.57 and Figure 4.58. The *RMSD* for the varying flow culvert inlet hydrograph was 0.35 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.59 and Figure 4.60. The *WSEL* profile upstream and downstream of the culvert for the varying and uniform flow models are a little less than 0.1 feet lower than unsteady *HEC-RAS*, due to the lower wetted perimeter value that is used in *FLO-2D* as described in Section 4.2.2.2 and Section 4.2.5.2. Refer to Table 4.20. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.59 and Figure 4.60. The *RMSD* for the varying maximum *WSEL* profile was 0.09 feet, which is a very minor deviation.

The results from Benchmark 4 are shown on Figure 4.61. The *FLO-2D WSEL* is slightly lower than *HEC-RAS* and there is minor numeric instability near stage 101, but the results match *HEC-RAS* very closely.

The results for this scenario are acceptable for *FCDMC* purposes.

Table 4.19 Scenario 3 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	102.92	95.73	39.91	0.23
HEC-RAS Varying	102.92	95.24	40.00	
FLO-2D Uniform	102.92	95.74	40.01	0.03
HEC-RAS Uniform	102.92	95.24	40.00	

Figure 4.56 Scenario 3 Rating Table Floodplain to Floodplain (OC) Test Results

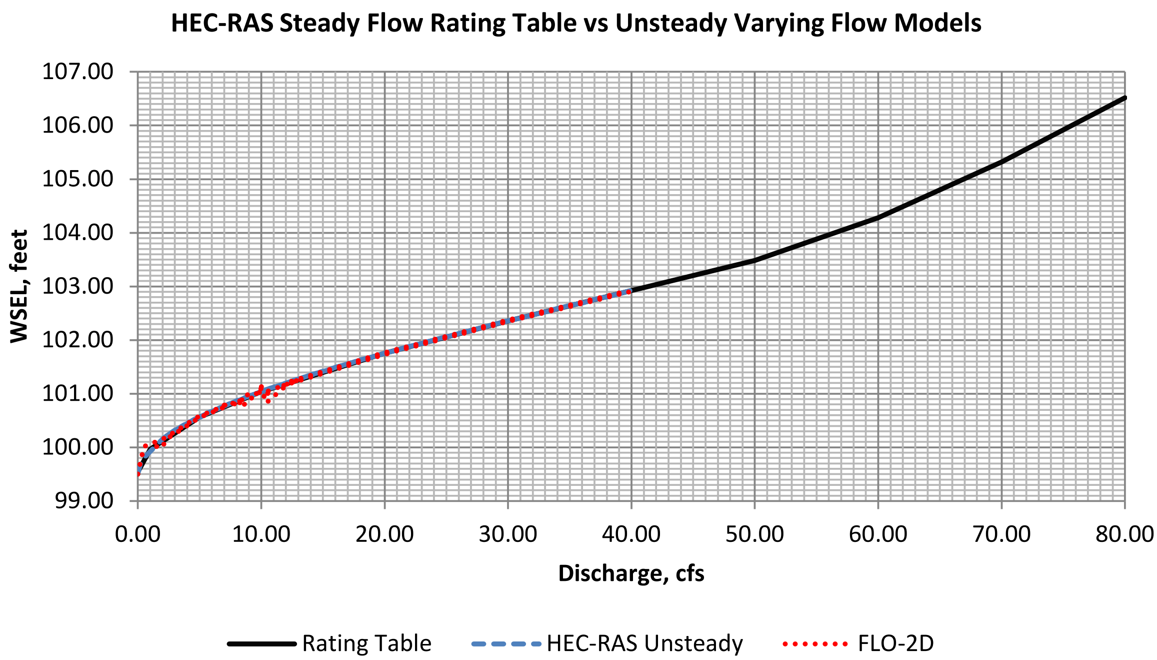


Figure 4.57 Scenario 3 Varying Flow Hydrograph Results

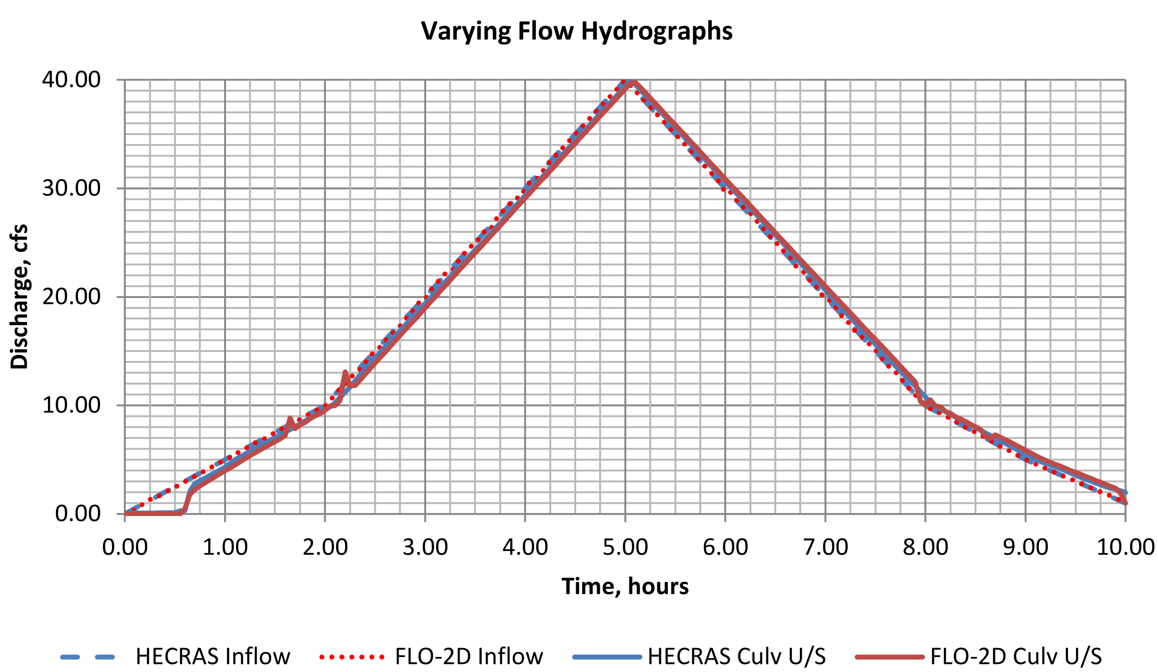


Figure 4.58 Scenario 3 Uniform Flow Hydrograph Results

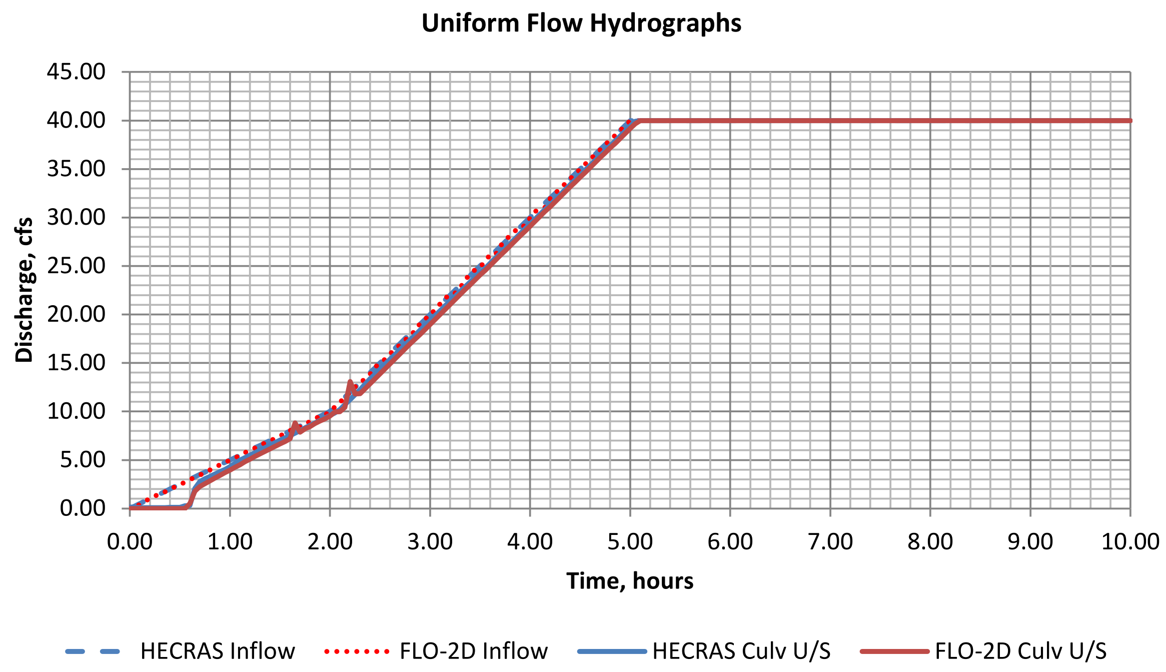


Figure 4.59 Scenario 3 Varying Flow *WSEL* Profile Results

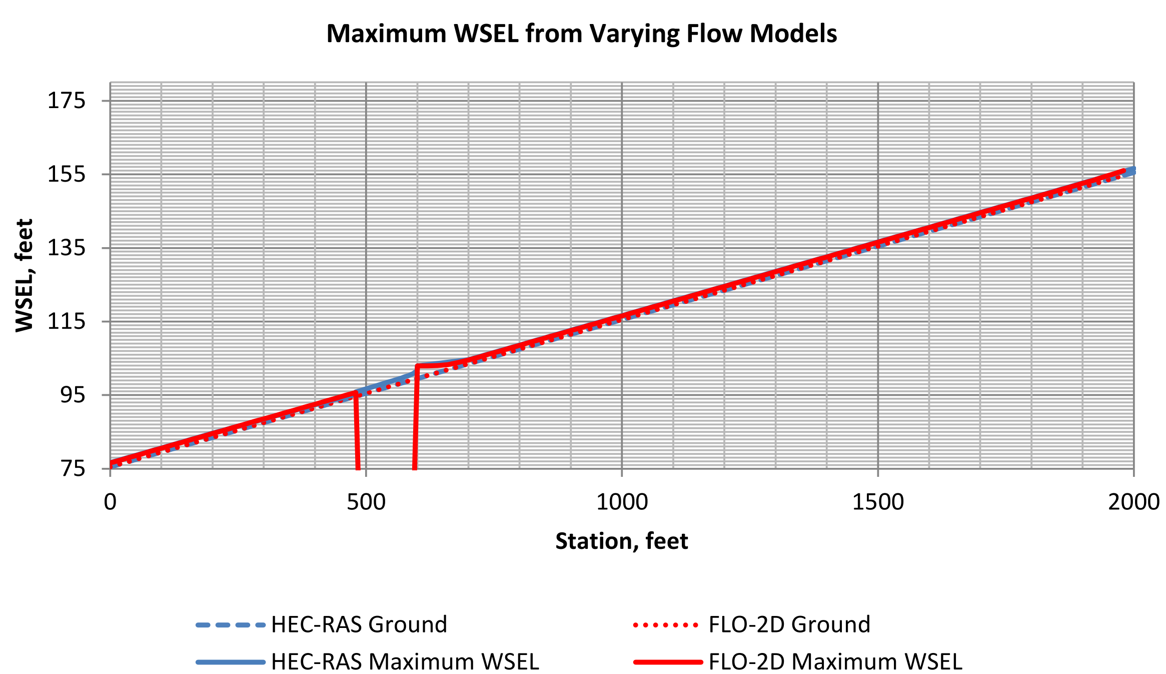


Figure 4.60 Scenario 3 Uniform Flow *WSEL* Profile Results

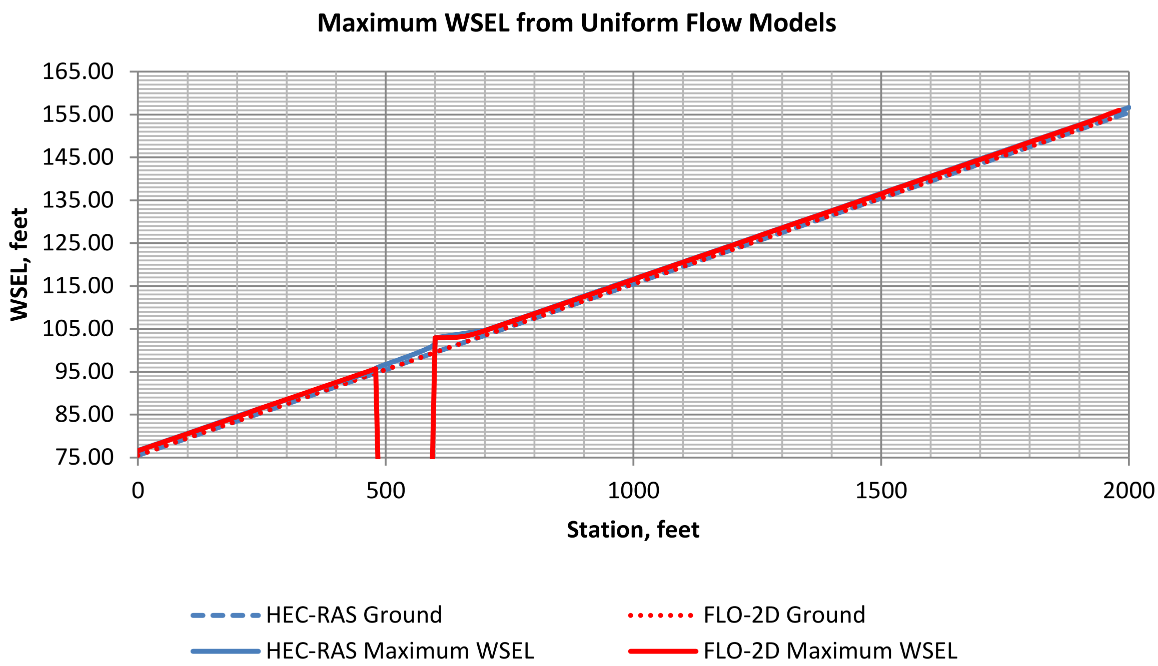
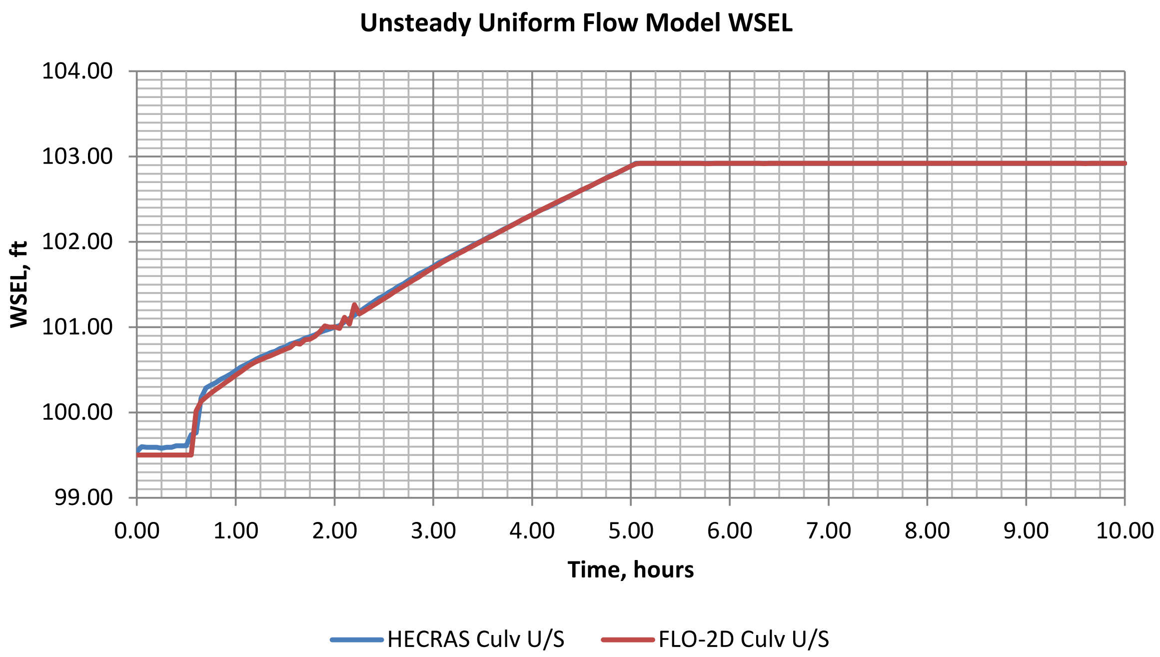


Table 4.20 Scenario 3 Comparison of Depth and Velocity at Station 2+00 (Grid 611)

***FLO-2D* (Time 8.95 hours)**					***HEC-RAS***				
Depth, ft	**Area, sf**	**Wetted Perimeter, ft**	***n***	**Velocity, fps**	**Depth, ft**	**Area, sf**	**Wetted Perimeter, ft**	***n***	**Velocity, fps**
1.04	8.59	8.28	0.065[1]	4.66	1.13	9.37	10.55	0.064	4.30
Computed by *FLO-2D*:				4.66	Computed by *HEC-RAS*:				4.27
Discharge by *FLO-2D*:				40.00	Discharge:				40.02
1	A limiting *Froude number* of 1.00 is applied to match the *HEC-RAS Froude number*. This results in virtually no *n*-value adjustments to force subcritical flow, which is why there is only a very small difference in *n*-value								
(yellow)	Computed using Manning's equation								

Figure 4.61 Scenario 3 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.5 Scenario 4: General Culvert Equations Floodplain to Floodplain (IC)

Scenario

Test of the *FLO-2D* general culvert equations approach for a culvert connecting two floodplain grids, operating under inlet control. The ground and culvert slopes were increased to 4.000% to force an inlet control condition.

Results and Discussion

Scenario 4 is the same as Scenario 3 except that the general culvert equations are used instead of a rating table. The results are compared with the unsteady *HEC-RAS* inlet control model. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.21. The maximum *WSEL* at the culvert inlet is an almost exact match with *HEC-RAS*. There is about a 4% difference in peak, which is acceptable. In general, differences of 10% of less in peak discharge are acceptable for *FCDMC* purposes. The maximum *WSEL* at the culvert outlet is 0.5 feet higher than *HEC-RAS*. *HEC-RAS* is reporting a much higher velocity leaving the culvert and *FLO-2D* is reporting the normal depth velocity in the channel. Immediately downstream of the culvert, the results agree for the remainder of the downstream reach. Upstream of the culvert, the results agree with unsteady *HEC-RAS*.

The results from Benchmark 1 are shown on Figure 4.62. The *FLO-2D* results match the steady state *HEC-RAS* rating table and unsteady *HEC-RAS* except for very slight numerical instability between 8 and 13 cfs. These instabilities disappear with a Courant setting of 0.1, but are not a concern for *FCDMC* purposes because they do not significantly affect the results. Refer to Section 4.2.5.19 for a discussion of the effects of applying other Courant values.

The results from Benchmark 2 are shown on Figure 4.63 and Figure 4.64. Although a little unstable between 8 and 13 cfs, the *FLO-2D* results compare very well with unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.63 and Figure 4.64. The *RMSD* for the varying flow culvert inlet hydrograph was 0.35 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.65 and Figure 4.66. The *WSEL* profile upstream and downstream of the culvert for the varying and uniform flow models are a little less than 0.1 feet lower than unsteady *HEC-RAS*, due to the lower wetted perimeter value that is used in *FLO-2D* as described in Section 4.2.2.2 and Section 4.2.5.2. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.65 and Figure 4.66. The *RMSD* for the varying maximum *WSEL* profile was 0.09 feet, which is a very minor deviation.

The results from Benchmark 4 are shown on Figure 4.67. The *FLO-2D WSEL* is slightly lower than *HEC-RAS* and there is minor numeric instability near stage 101, but the results match *HEC-RAS* very closely.

The results for this scenario are acceptable for *FCDMC* purposes.

Table 4.21 Scenario 4 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	102.88	95.74	41.65	4.13
HEC-RAS Varying	102.92	95.24	40.00	
FLO-2D Uniform	102.88	95.74	41.66	4.15
HEC-RAS Uniform	102.92	95.24	40.00	

Figure 4.62 Scenario 4 Rating Table Floodplain to Floodplain (OC) Test Results

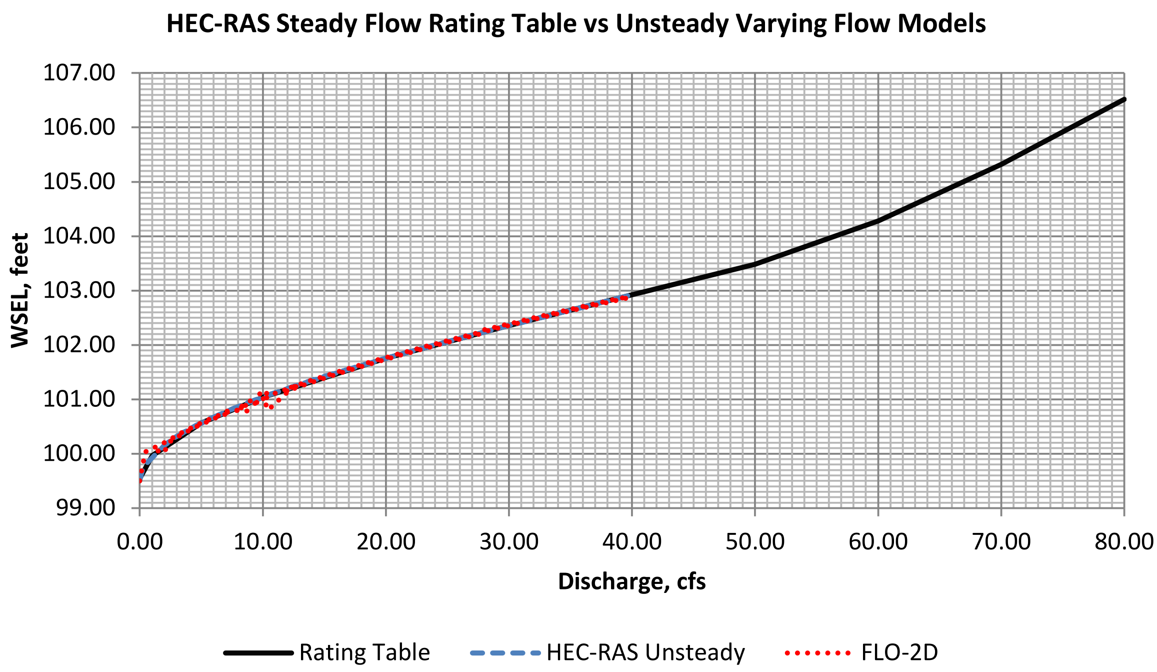


Figure 4.63 Scenario 4 Varying Flow Hydrograph Results

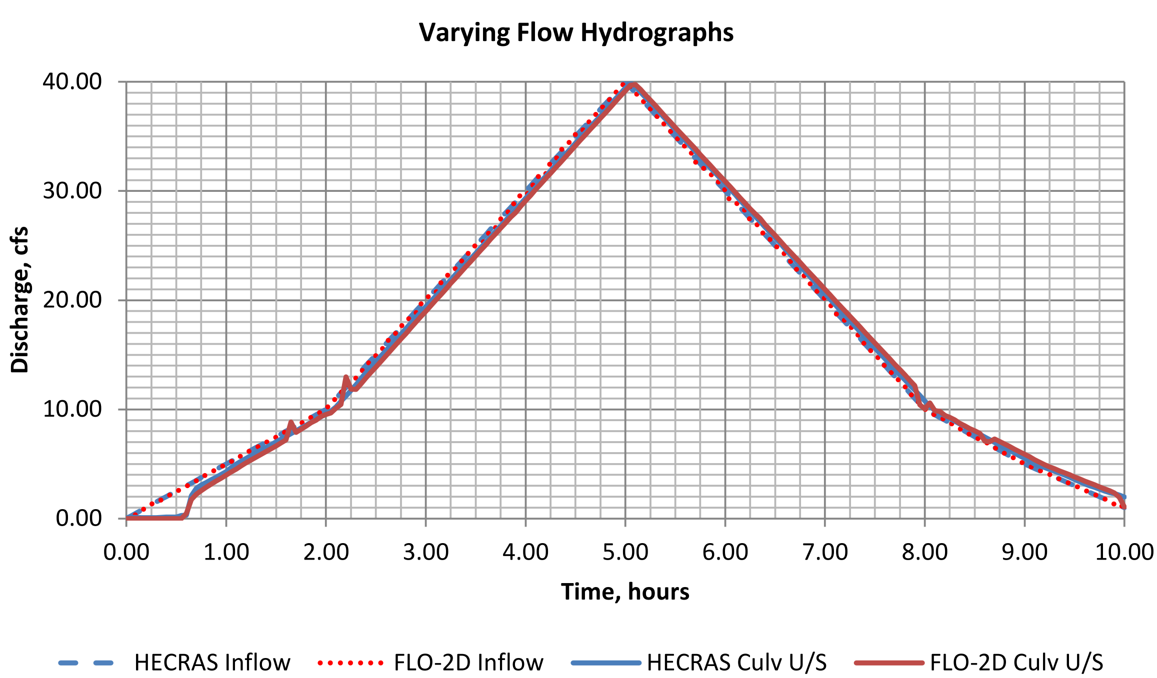


Figure 4.64 Scenario 4 Uniform Flow Hydrograph Results

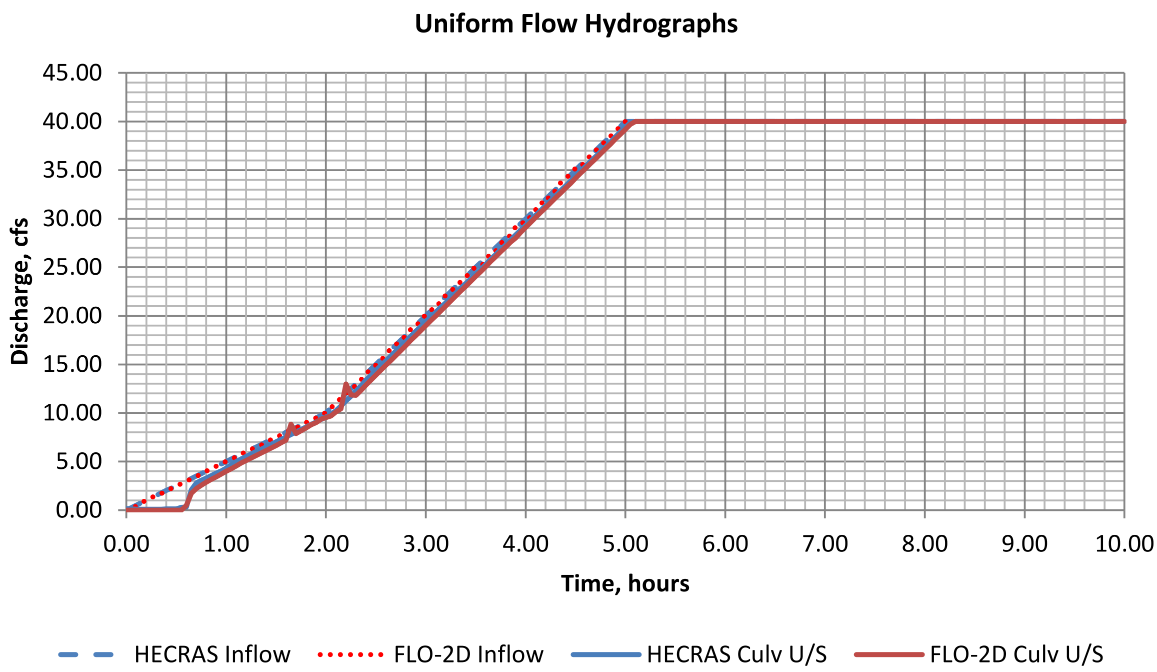


Figure 4.65 Scenario 4 Varying Flow *WSEL* Profile Results

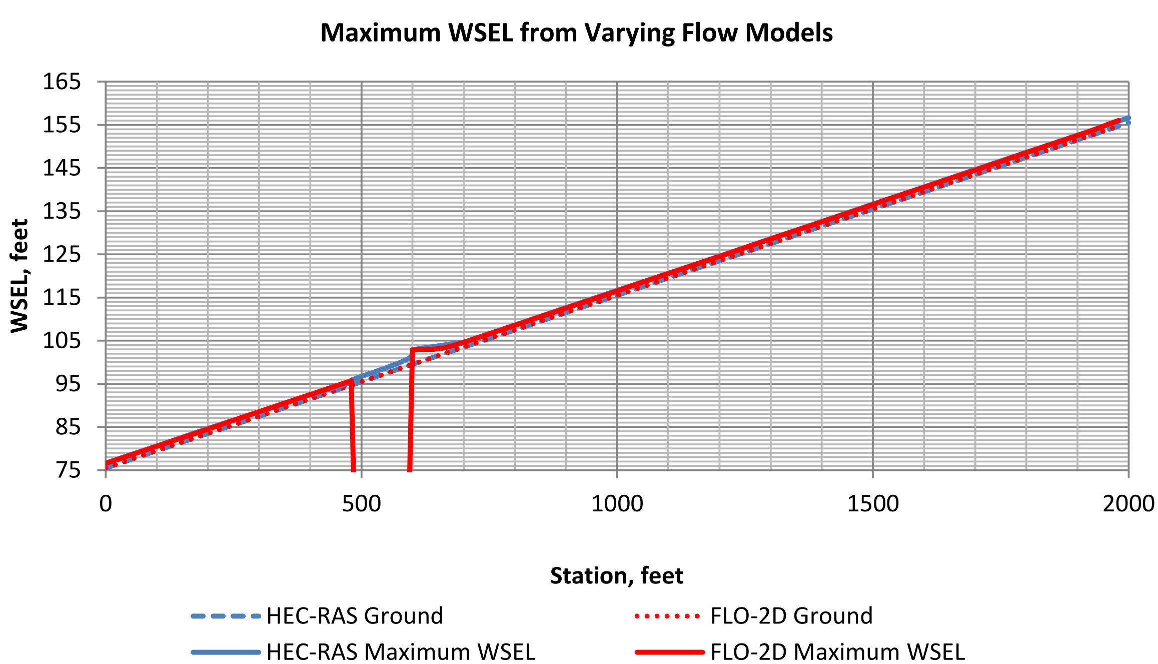


Figure 4.66 Scenario 4 Uniform Flow *WSEL* Profile Results

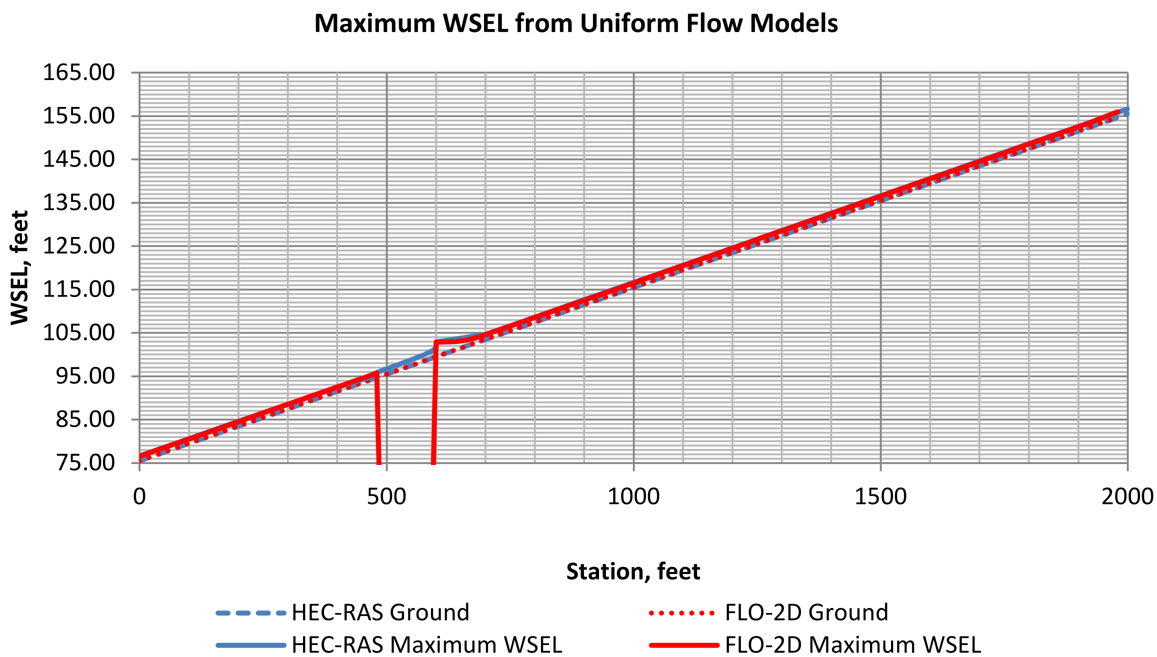
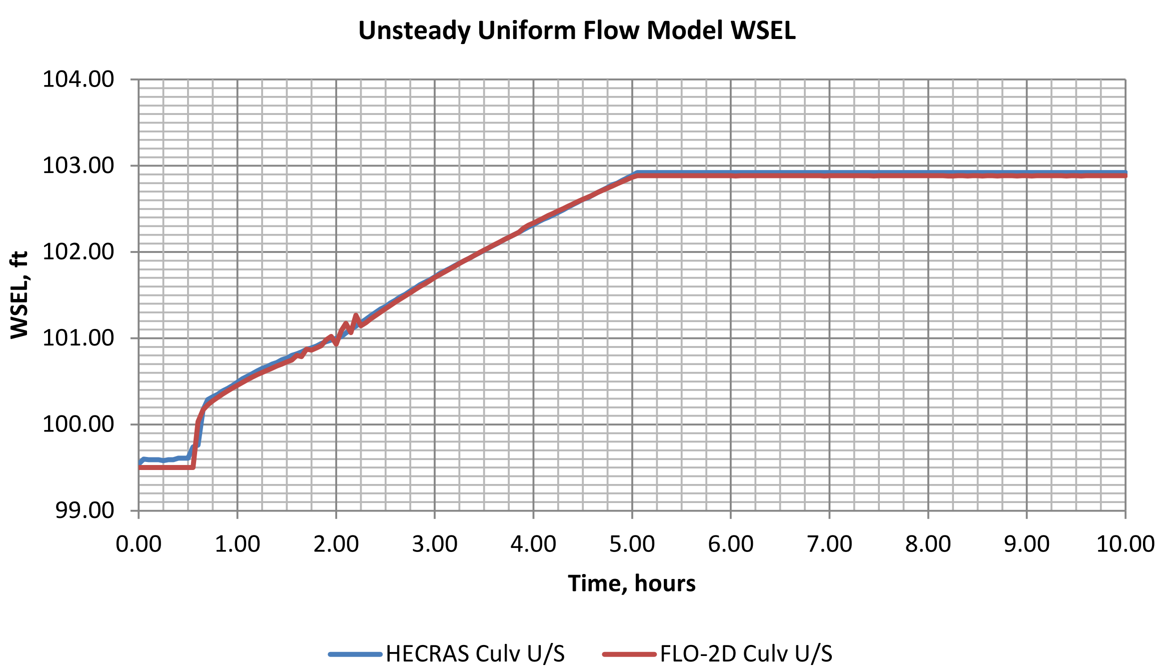


Figure 4.67 Scenario 4 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.6 Scenario 5: Rating Table Floodplain to Floodplain (IC and OC)

Scenario

Test of the *FLO-2D* rating table approach for a culvert connecting two floodplain grids, switching between inlet and outlet control. The ground and culvert slopes were changed to 0.450% to force an inlet/outlet control condition. The *FLO-2D* INOUTCONT variable was set to 0 to force no adjustment due to tailwater.

Results and Discussion

Scenario 5 is the same as Scenario 1 and Scenario 3 except that the ground and culvert slope was changed to 0.450%. The results are compared with the unsteady *HEC-RAS* inlet/outlet control model. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.22. The maximum *WSEL* at the culvert inlet is a very close match with *HEC-RAS*. The maximum *FLO-2D WSEL* at the culvert outlet is 0.4 to 0.5 feet lower than *HEC-RAS*. Immediately downstream of the culvert, the *FLO-2D* results maintain the 0.4 foot difference for the remainder of the downstream reach, as expected due to the difference in wetted perimeter. Refer to Table 4.23, which represents results from the uniform flow model. Upstream of the culvert, the *FLO-2D* results are slightly higher than unsteady *HEC-RAS* from the culvert inlet upstream to about station 1200, and then the difference gradually increases until the *FLO-2D* results are again.0.4 feet lower than *HEC-RAS*.

The results from Benchmark 1 are shown on Figure 4.68. The *FLO-2D* results match steady the state *HEC-RAS* rating table and unsteady *HEC-RAS* very well.

The results from Benchmark 2 are shown on Figure 4.69 and Figure 4.70. The *FLO-2D* results compare very well with unsteady *HEC-RAS*, although there is slight instability at peak. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.69 and Figure 4.70. The *RMSD* for the varying flow culvert inlet hydrograph was 0.51 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.71 and Figure 4.72. The *WSEL* profile upstream and downstream of the culvert (outside of areas influenced by backwater) for the varying and uniform flow models are about 0.4 feet lower than unsteady *HEC-RAS* as described above, due to the lower wetted perimeter value that is used in *FLO-2D* as described in Section 4.2.2.2 and Section 4.2.5.2. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.71 and Figure 4.72. The *RMSD* for the varying maximum *WSEL* profile was 0.33 feet, which checks with the visual observation.

The results from Benchmark 4 are shown on Figure 4.73. The *FLO-2D WSEL* is slightly higher than *HEC-RAS* and there is numeric instability after the uniform flow peak discharge is reached, but the results match *HEC-RAS* reasonably well.

The results for this scenario are acceptable for *FCDMC* purposes.

<table>
<tr><th colspan="5">Table 4.22 Scenario 5 Comparison of Hydraulic Structure Model Results</th></tr>
<tr><th rowspan="2">Model</th><th colspan="2">Water Surface Elevation at Peak</th><th rowspan="2">Discharge through Culvert, cfs</th><th rowspan="2">Percent Difference</th></tr>
<tr><th>Inlet</th><th>Outlet</th></tr>
<tr><td>FLO-2D Varying</td><td>82.94</td><td>79.92</td><td>58.60</td><td rowspan="2">0.81</td></tr>
<tr><td>HEC-RAS Varying</td><td>82.95</td><td>80.33</td><td>58.13</td></tr>
<tr><td>FLO-2D Uniform</td><td>83.27</td><td>79.98</td><td>61.78</td><td rowspan="2">2.97</td></tr>
<tr><td>HEC-RAS Uniform</td><td>83.16</td><td>80.38</td><td>60.00</td></tr>
</table>

<table>
<tr><th colspan="10">Table 4.23 Scenario 5 Comparison of Depth and Velocity at Station 2+00 (Grid 611)</th></tr>
<tr><th colspan="5">FLO-2D at Time 8.59 hours</th><th colspan="5">HEC-RAS</th></tr>
<tr><th>Depth, ft</th><th>Area, sf</th><th>Wetted Perimeter, ft</th><th>n</th><th>Velocity, fps</th><th>Depth, ft</th><th>Area, sf</th><th>Wetted Perimeter, ft</th><th>n</th><th>Velocity, fps</th></tr>
<tr><td>2.31</td><td>19.14</td><td>8.28</td><td>0.055</td><td>3.16</td><td>2.63</td><td>21.79</td><td>13.54</td><td>0.053</td><td>2.66</td></tr>
<tr><td colspan="4">Computed by FLO-2D:</td><td>3.17</td><td colspan="4">Computed by HEC-RAS:</td><td>2.66</td></tr>
<tr><td colspan="4">Discharge by FLO-2D:</td><td>60.78</td><td colspan="4">Discharge:</td><td>57.95</td></tr>
<tr><td></td><td colspan="9">Computed using Manning’s equation</td></tr>
</table>

Figure 4.68 Scenario 5 Rating Table Floodplain to Floodplain (OC) Test Results

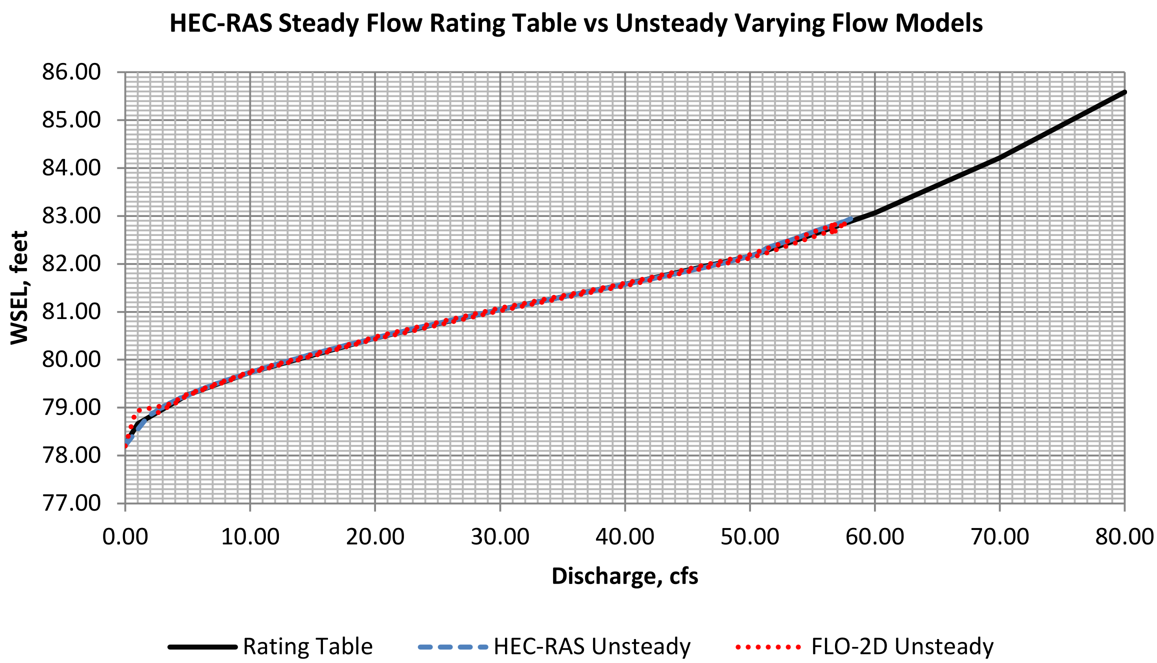


Figure 4.69 Scenario 5 Varying Flow Hydrograph Results

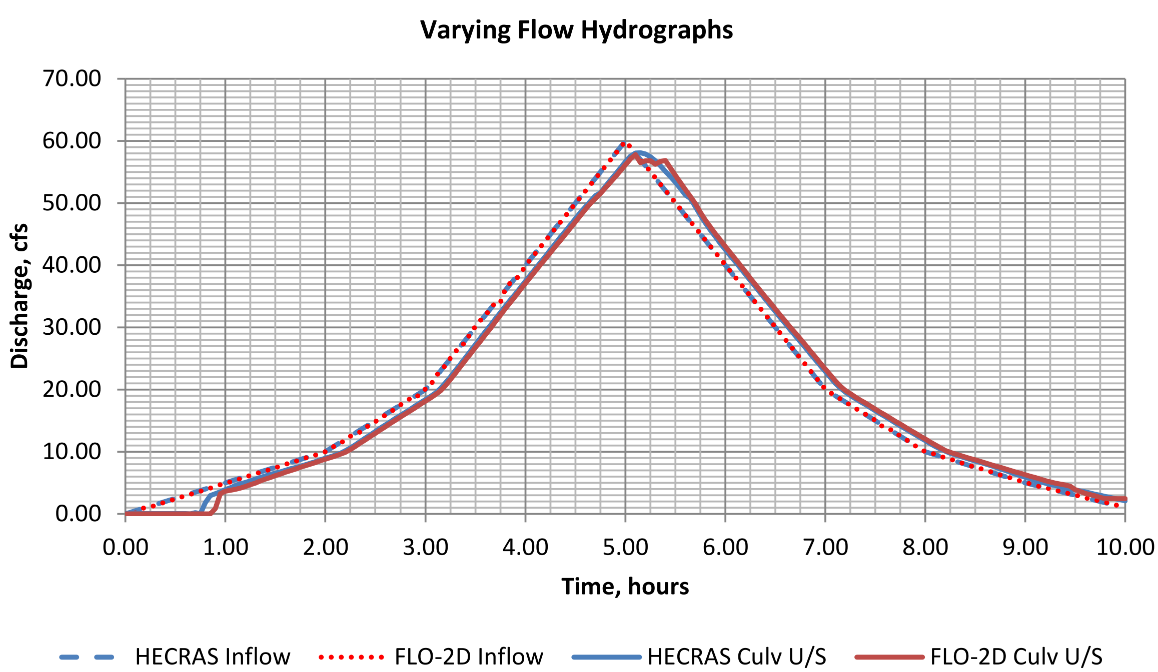


Figure 4.70 Scenario 5 Uniform Flow Hydrograph Results

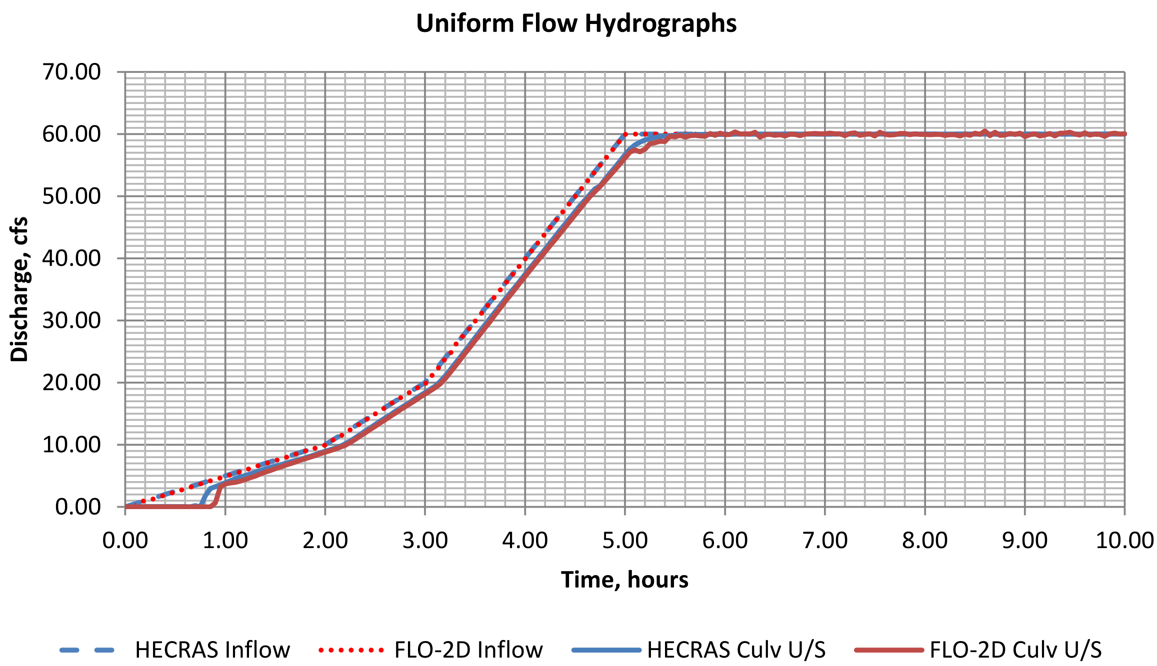


Figure 4.71 Scenario 5 Varying Flow *WSEL* Profile Results

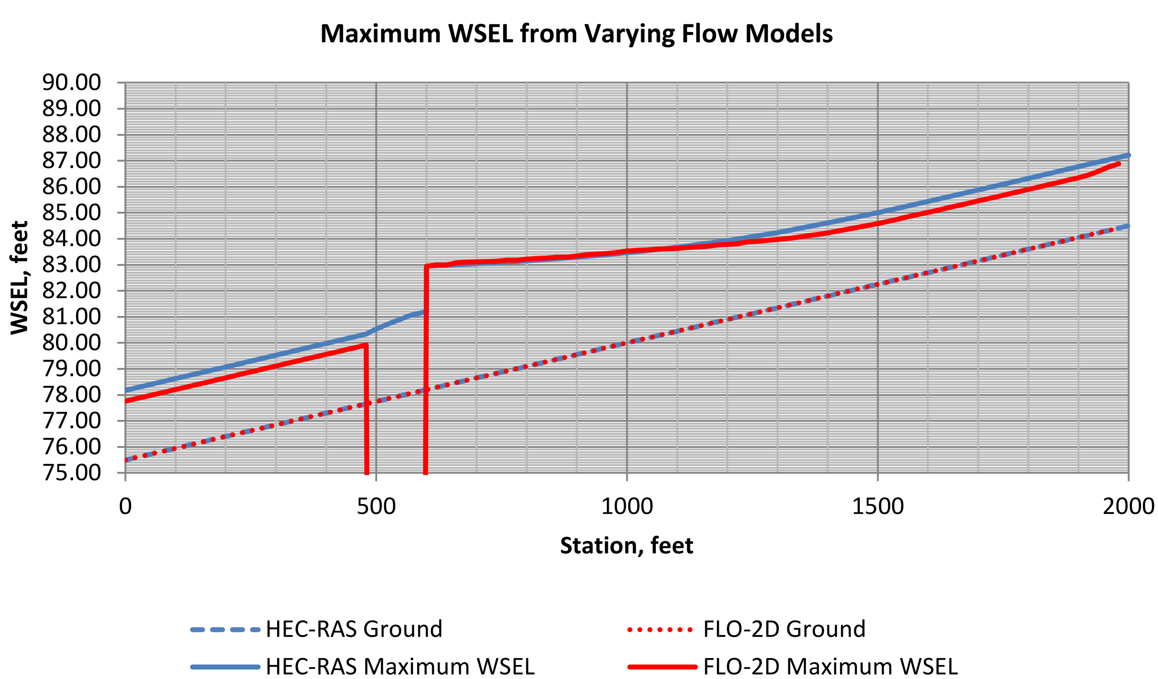


Figure 4.72 Scenario 5 Uniform Flow *WSEL* Profile Results

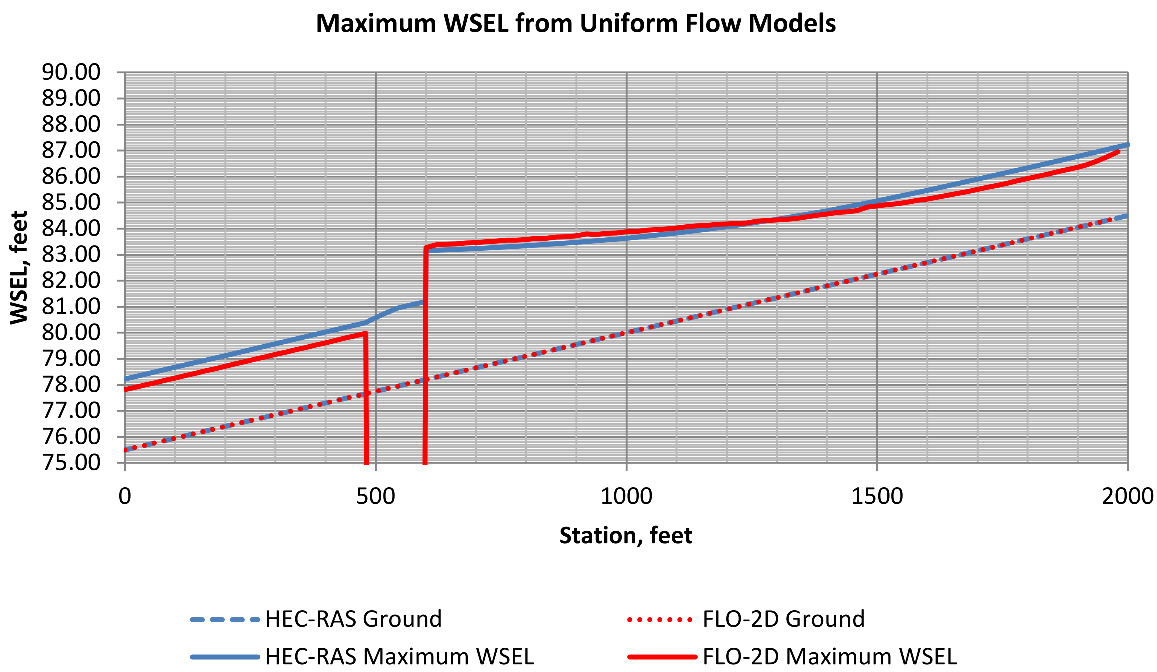
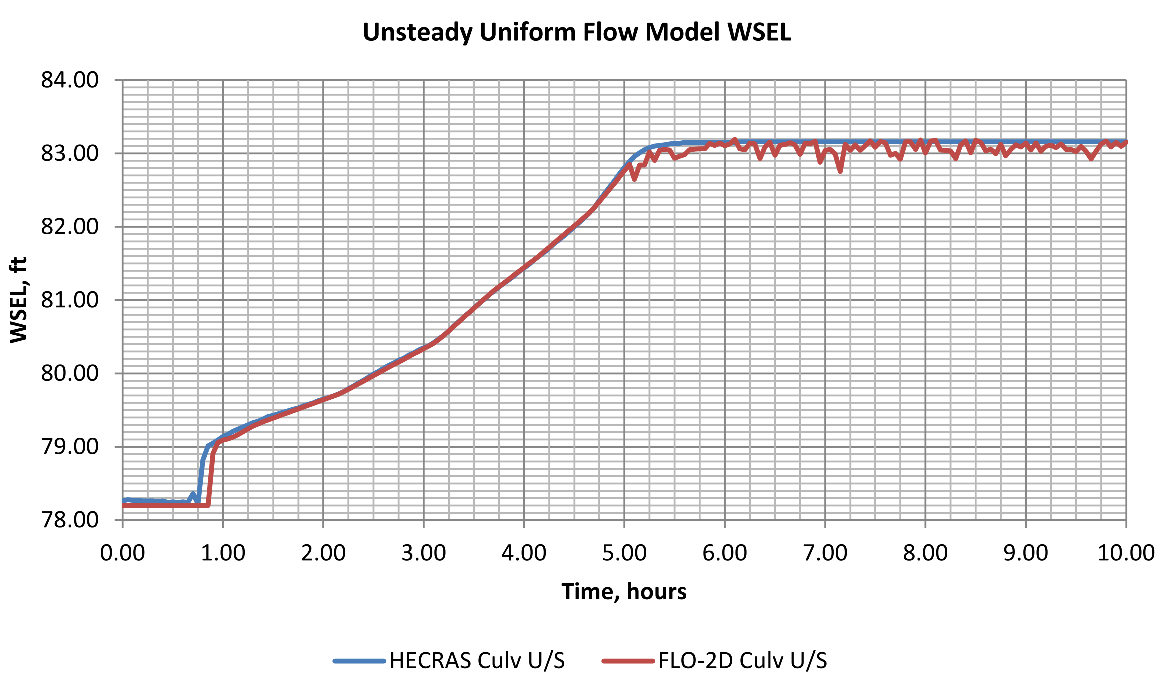


Figure 4.73 Scenario 5 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.7 Scenario 6: General Culvert Equations Floodplain to Floodplain (IC and OC)

Scenario

Test of the *FLO-2D* general culvert equations approach for a culvert connecting two floodplain grids, switching between inlet and outlet control. The ground and culvert slopes were changed to 0.450% to force an inlet/outlet control condition.

Results and Discussion

Scenario 6 is the same as Scenario 5 except that the general culvert equations option is applied. The results are compared with the unsteady *HEC-RAS* inlet/outlet control model. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.24. The maximum *WSEL* at the culvert inlet is a close match with *HEC-RAS*, better than the rating table model results. The maximum *FLO-2D WSEL* at the culvert outlet is 0.4 to 0.5 feet lower than *HEC-RAS*. Immediately downstream of the culvert, the *FLO-2D* results maintain the 0.4 foot difference for the remainder of the downstream reach, as expected due to the difference in wetted perimeter. Refer to Table 4.25, which represents results from the uniform flow model. Upstream of the culvert, the *FLO-2D* results are slightly higher than unsteady *HEC-RAS* from the culvert inlet upstream to about station 1200 due to backwater effects, and then the difference gradually increases until the *FLO-2D* results are again.0.4 feet lower than *HEC-RAS*.

The results from Benchmark 1 are shown on Figure 4.74. The *FLO-2D* results match the steady state *HEC-RAS* rating table and unsteady *HEC-RAS* very well.

The results from Benchmark 2 are shown on Figure 4.75 and Figure 4.76. The *FLO-2D* results compare very well with unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.75 and Figure 4.76. The *RMSD* for the varying flow culvert inlet hydrograph was 0.47 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.77 and Figure 4.78. The *WSEL* profile upstream and downstream of the culvert for the varying and uniform flow models are nominally about 0.4 feet lower than unsteady *HEC-RA* as described above, due to the lower wetted perimeter value that is used in *FLO-2D* as described in Section 4.2.2.2 and Section 4.2.5.2. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.77 and Figure 4.78. The *RMSD* for the varying maximum *WSEL* profile was 0.37 feet, which checks with the visual observation.

The results from Benchmark 4 are shown on Figure 4.79. The *FLO-2D WSEL* is slightly lower than *HEC-RAS* and there is slight numeric instability after the uniform flow peak discharge is reached, but the results match *HEC-RAS* reasonably well.

The results for this scenario are acceptable for *FCDMC* purposes.

Table 4.24 Scenario 6 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	82.86	79.93	58.44	0.53
HEC-RAS Varying	82.95	80.33	58.13	
FLO-2D Uniform	83.15	79.99	62.34	3.92
HEC-RAS Uniform	83.16	80.38	60.00	

Table 4.25 Scenario 6 Comparison of Depth and Velocity at Station 2+00 (Grid 611)

***FLO-2D* at Time 5.14 hours**					***HEC-RAS***				
Depth, ft	**Area, sf**	**Wetted Perimeter, ft**	**n**	**Velocity, fps**	**Depth, ft**	**Area, sf**	**Wetted Perimeter, ft**	**n**	**Velocity, fps**
2.32	19.32	8.28	0.055	3.17	2.63	21.79	13.54	0.053	2.66
Computed by *FLO-2D*:				3.18	Computed by *HEC-RAS*:				2.66
Discharge by *FLO-2D*:				61.07	Discharge:				57.95
	Computed using Manning's equation								

Figure 4.74 Scenario 6 Rating Table Floodplain to Floodplain (IC and OC) Test Results

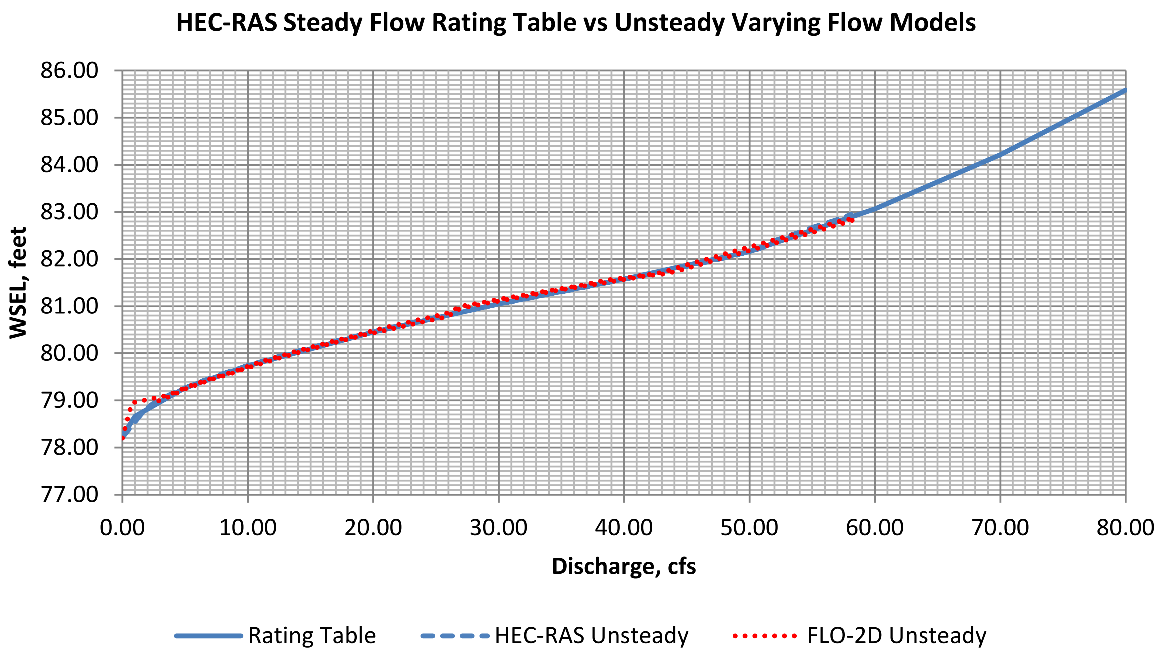


Figure 4.75 Scenario 6 Varying Flow Hydrograph Results

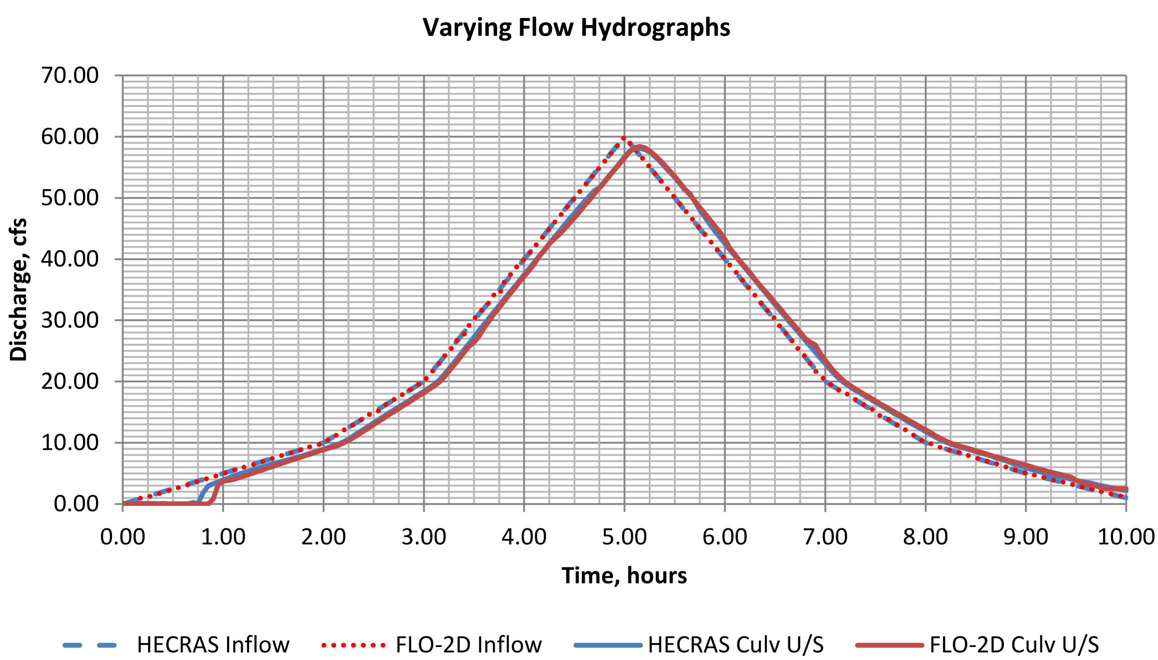


Figure 4.76 Scenario 6 Uniform Flow Hydrograph Results

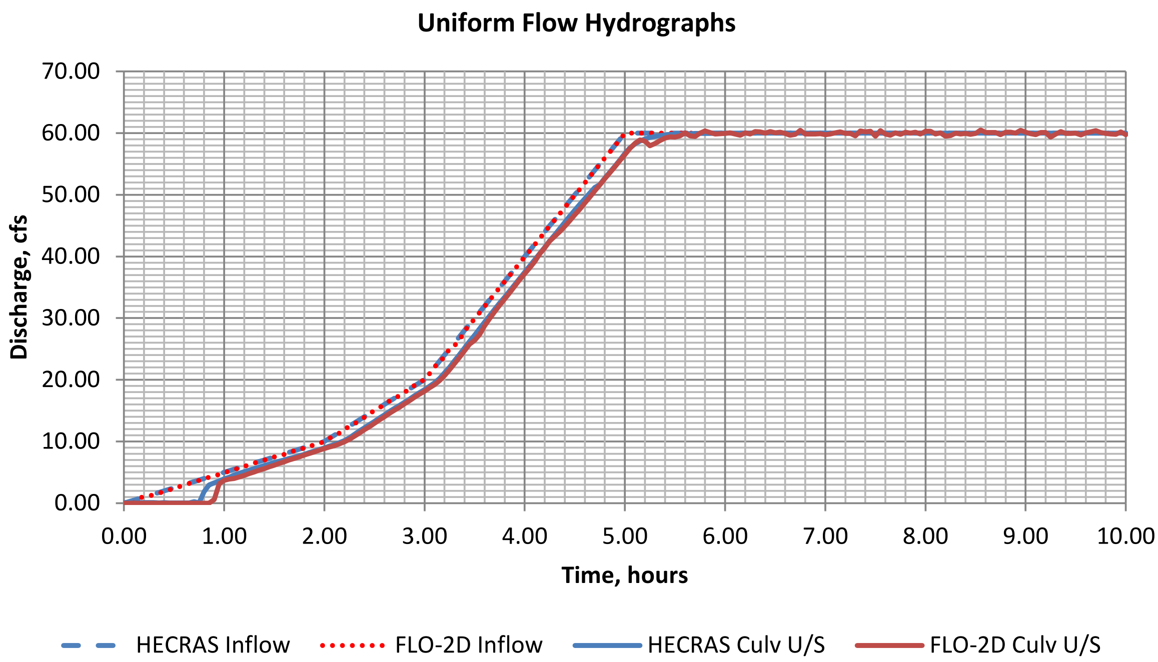


Figure 4.77 Scenario 6 Varying Flow *WSEL* Profile Results

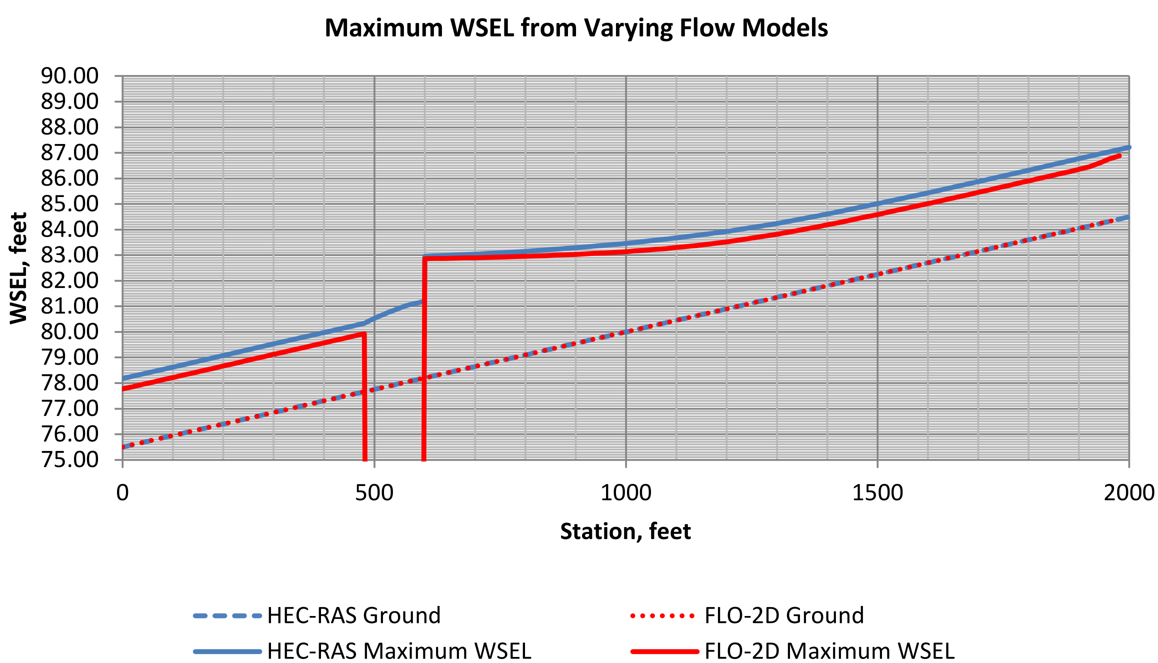


Figure 4.78 Scenario 6 Uniform Flow *WSEL* Profile Results

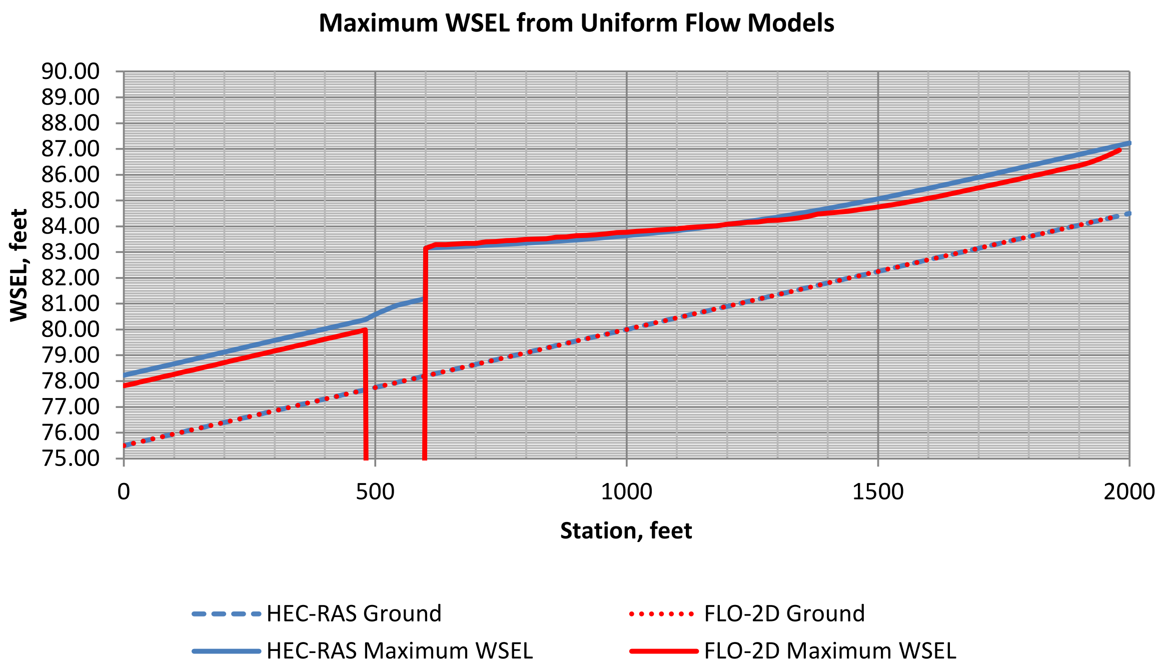
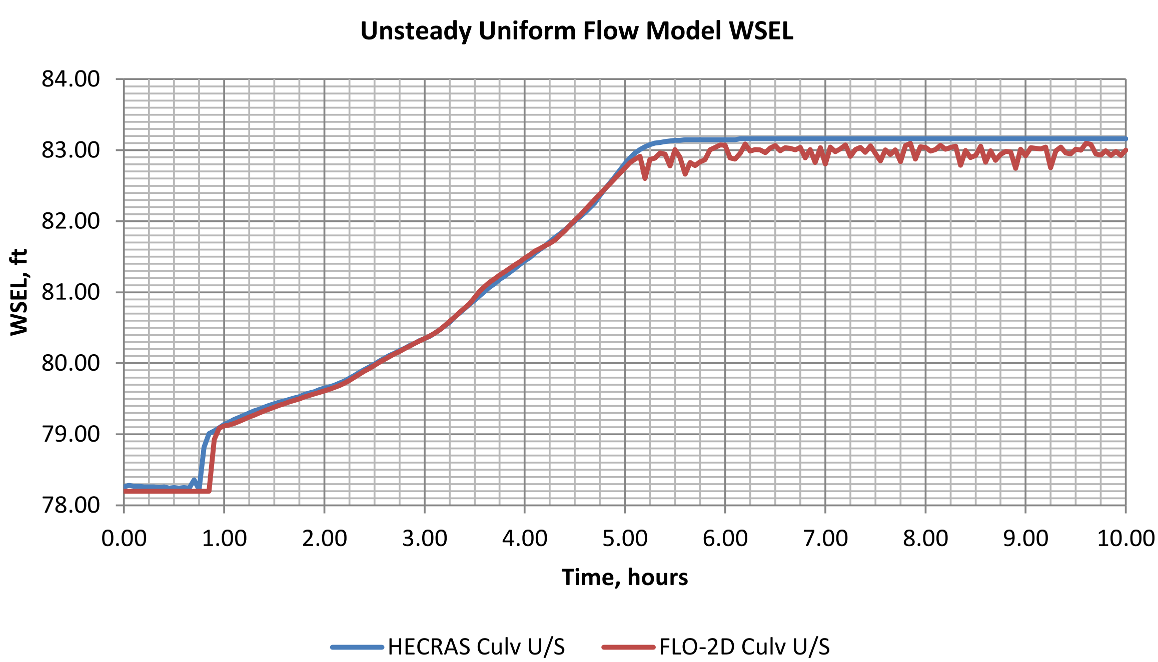


Figure 4.79 Scenario 6 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.8 Scenario 7: Rating Table Channel to Channel (OC)

Scenario

Test of the *FLO-2D* rating table approach for a culvert connecting two *1D* channel grids, operating under outlet control.

Results and Discussion

Scenario 7 is the same as Scenario 1 except that the *1D* channel option was used for the entire length of the model instead of a channel defined using floodplain grids. The culvert was assigned to connect a *1D* channel element to a *1D* channel element. The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.26. The *FLO-2D* results for the culvert at peak flow are nearly identical to unsteady *HEC-RAS* at the inlet and outlet. A check of channel hydraulics based on depth-variable Manning's *n* is shown in Table 4.27 for the varying flow hydrograph condition at Station 2+00 (Grid 611). The *FLO-2D* results are identical to unsteady *HEC-RAS*.

The results from Benchmark 1 are shown on Figure 4.80. The rating table based results are very stable and are nearly identical to the rating table generated using steady state *HEC-RAS* and the unsteady *HEC-RAS* results.

The results from Benchmark 2 are shown on Figure 4.81 and Figure 4.82. The *FLO-2D* results are very stable and nearly identical to unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.81 and Figure 4.82. The *RMSD* for the varying flow culvert inlet hydrograph was 0.37 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.83 and Figure 4.84. The *WSEL* profiles downstream and upstream of the culvert for the varying and uniform flow models are nearly identical to *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.83 and Figure 4.84. The *RMSD* for the varying maximum *WSEL* profile was 0.02 feet, which is a very minor deviation.

The results from Benchmark 4 are shown on Figure 4.85. The *FLO-2D WSEL* are nearly identical to *HEC-RAS*.

The *FLO-2D* results for Scenario 7 are a very close match with unsteady *HEC-RAS* and are acceptable for *FCDMC* purposes.

Table 4.26 Scenario 7 Comparison of Hydraulic Structure Model Results				
Model	**Water Surface Elevation at Peak**		**Discharge through Culvert, cfs**	**Percent Difference**
	Inlet	**Outlet**		
FLO-2D Varying	80.60	79.55	37.83	0.45
HEC-RAS Varying	80.58	79.54	37.66	
FLO-2D Uniform	80.83	79.67	40.00	0.00
HEC-RAS Uniform	80.85	79.67	40.00	

Table 4.27 Scenario 7 Comparison of Depth and Velocity at Station 2+00 (Grid 611)									
***FLO-2D* at Time 9.77 hours**					***HEC-RAS***				
Depth, ft	**Area, sf**	**Wetted Perimeter, ft**	**n**	**Velocity, fps**	**Depth, ft**	**Area, sf**	**Wetted Perimeter, ft**	**n**	**Velocity, fps**
3.21	26.59	14.70	0.066	1.50	3.21	26.63	14.70	0.066	1.51
Computed by *FLO-2D*:				1.50	Computed by *HEC-RAS*:				1.50
Discharge by *FLO-2D*:				40.00	Discharge:				40.00
	Computed using Manning’s equation								

Figure 4.80 Scenario 7 Rating Table Floodplain to Floodplain (OC) Test Results

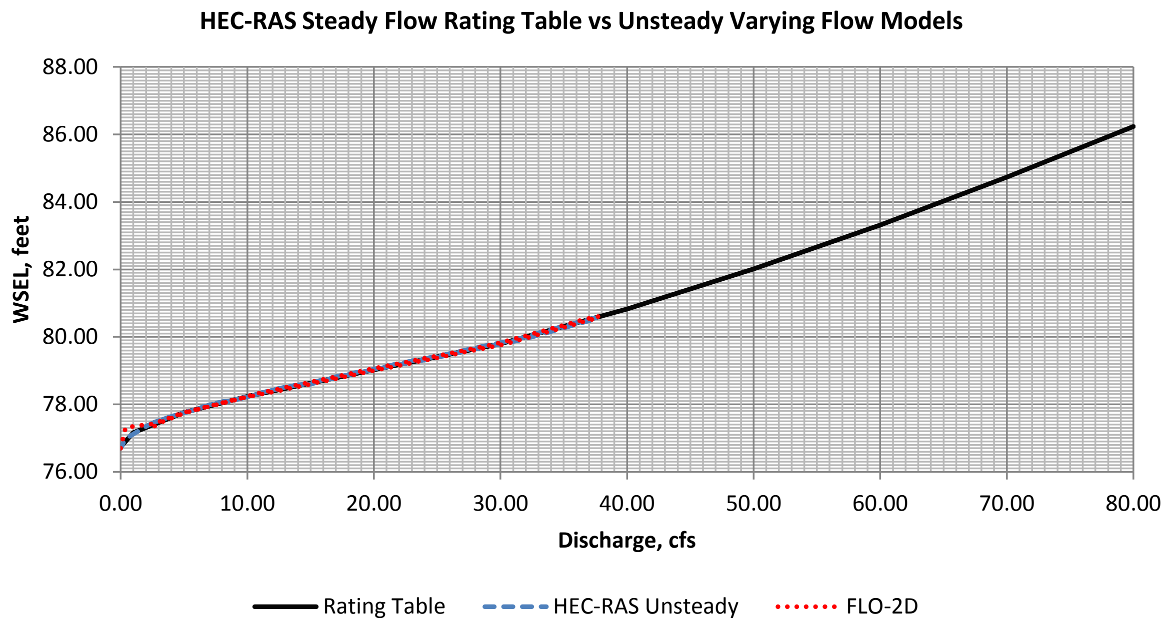


Figure 4.81 Scenario 7 Varying Flow Hydrograph Results

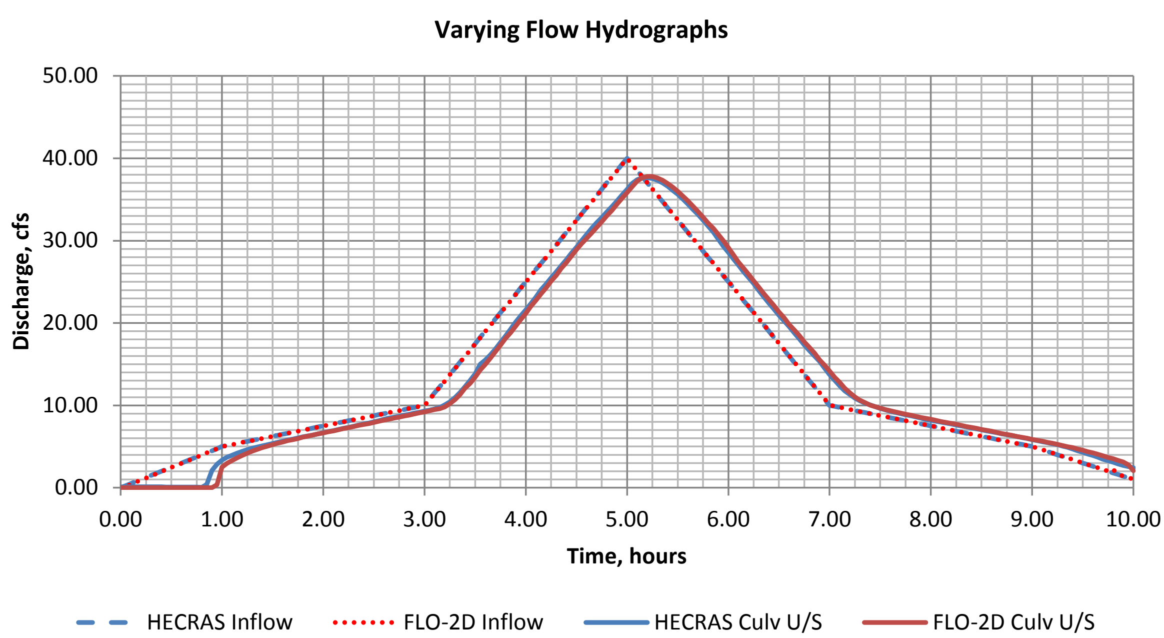


Figure 4.82 Scenario 7 Uniform Flow Hydrograph Results

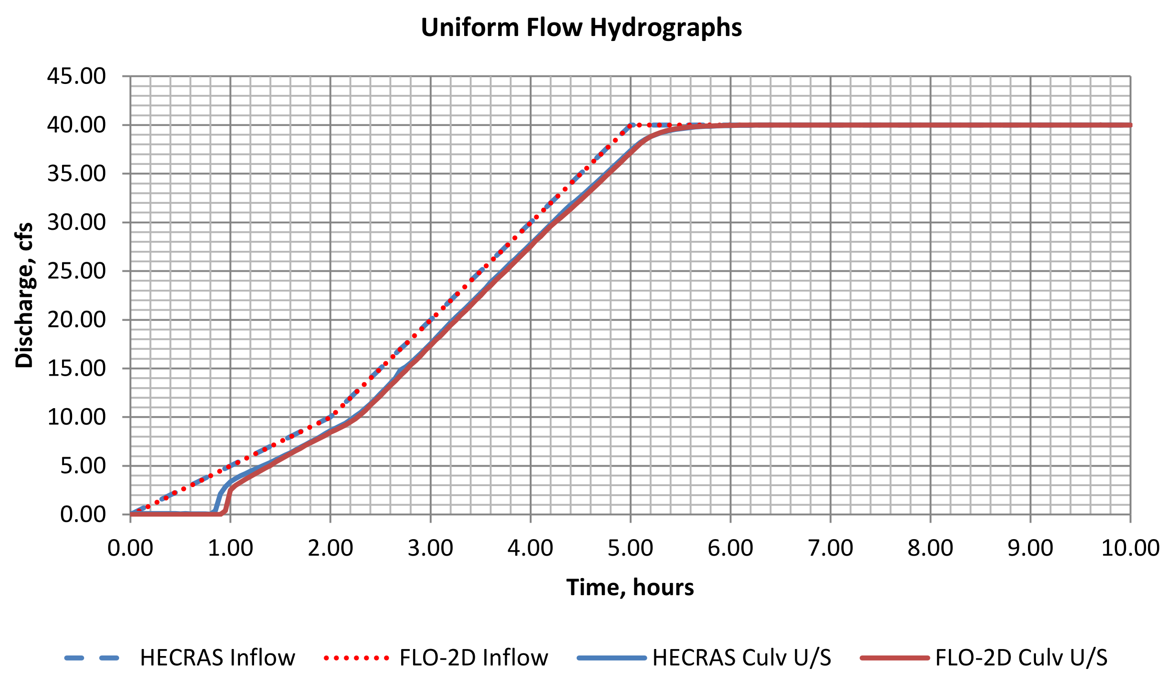


Figure 4.83 Scenario 7 Varying Flow *WSEL* Profile Results

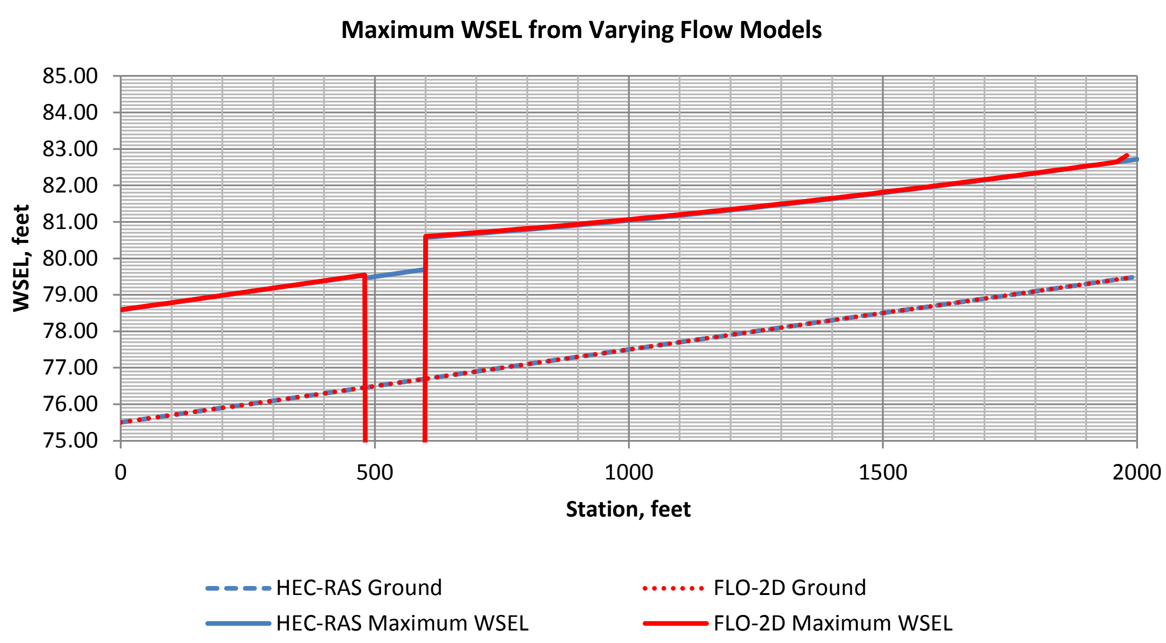


Figure 4.84 Scenario 7 Uniform Flow *WSEL* Profile Results

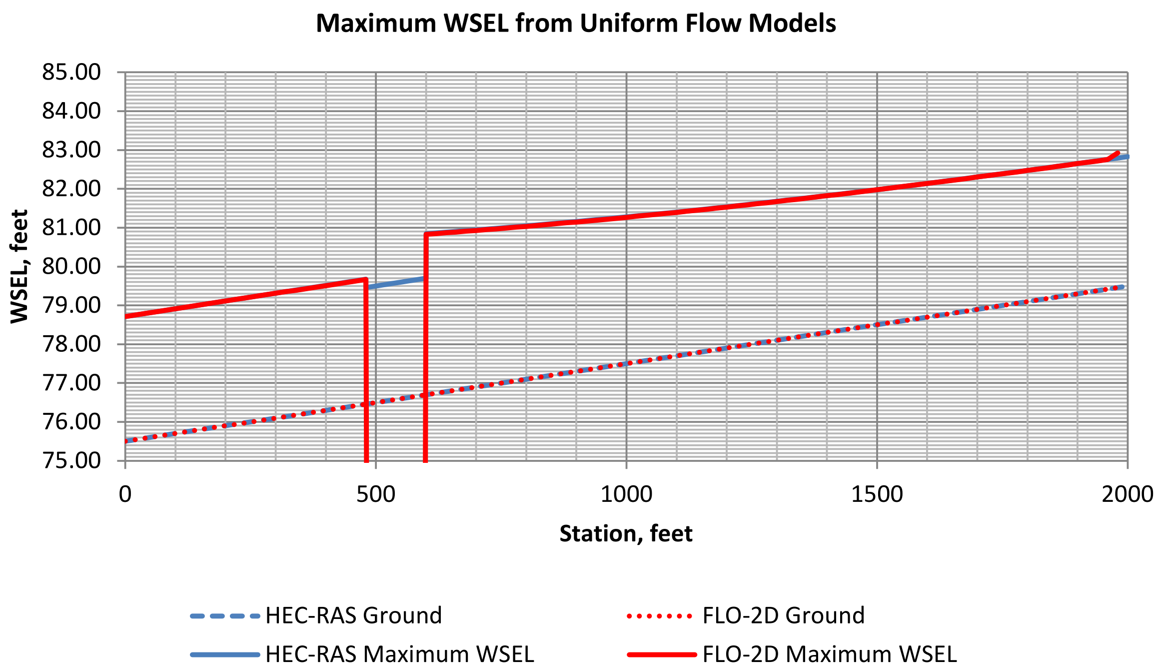
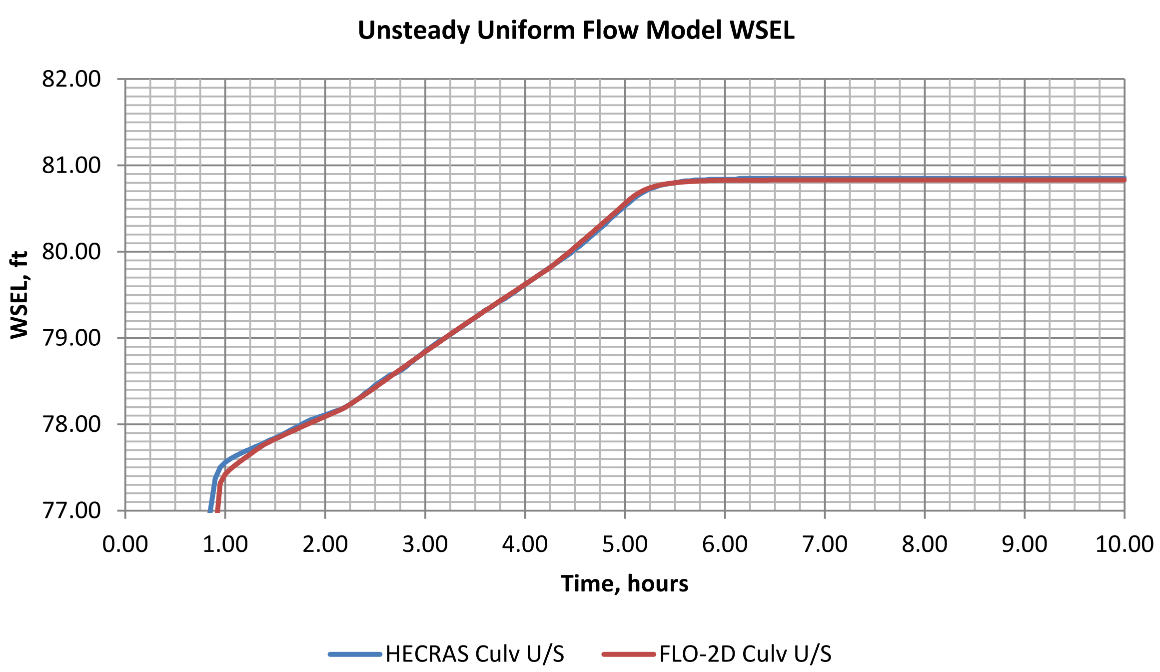


Figure 4.85 Scenario 7 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.9 Scenario 8: General Culvert Equations Channel to Channel (OC)

Scenario

Test of the *FLO-2D* general culvert equations approach for a culvert connecting two *1D* channel elements, operating under outlet control.

Results and Discussion

Scenario 8 is the same as Scenario 7 except that the general culvert equations option was used instead of the rating table option**.** The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.28. The *FLO-2D* varying and uniform flow results for the culvert at peak flow are 0.06 feet and 0.11 feet lower, respectively, than unsteady *HEC-RAS* at the inlet; while at the outlet, both the varying and uniform flow results are 0.22 feet lower than unsteady *HEC-RAS*.

The results from Benchmark 1 are shown on Figure 4.86. The general culvert equation-based results are a little unsteady near peak; otherwise, they are very close to the rating table generated using steady state *HEC-RAS* and the unsteady *HEC-RAS* results.

The results from Benchmark 2 are shown on Figure 4.87 and Figure 4.88. The *FLO-2D* results are slightly unstable near peak but compare very well with unsteady *HEC-RAS* for both the varying and uniform flow hydrograph conditions. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.87 and Figure 4.88. The *RMSD* for the varying flow culvert inlet hydrograph was 0.39 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.89 and Figure 4.90. The *FLO-2D WSEL* profiles downstream and upstream of the culvert for the varying and uniform flow models are slightly lower than *HEC-RAS* at the culvert inlet and outlet, as noted above. Upstream and downstream of the culvert, the *FLO-2D* channel hydraulics results match unsteady *HEC-RAS* very closely. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.89 and Figure 4.90. The *RMSD* for the varying maximum *WSEL* profile was 0.03 feet, which is a very minor deviation.

The results from Benchmark 4 are shown on Figure 4.91. The *FLO-2D WSEL* is a little lower than unsteady *HEC-RAS* as described above.

The minor differences between the *FLO-2D* and unsteady *HEC-RAS* results for Scenario 8 are acceptable for *FCDMC* purposes, although the rating table approach provides improved results (refer to Scenario 7).

Table 4.28 Scenario 8 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	80.51	79.32	38.29	1.67
HEC-RAS Varying	80.58	79.54	37.66	
FLO-2D Uniform	80.74	79.45	40.01	0.03
HEC-RAS Uniform	80.85	79.67	40.00	

Figure 4.86 Scenario 8 Rating Table Floodplain to Floodplain (OC) Test Results

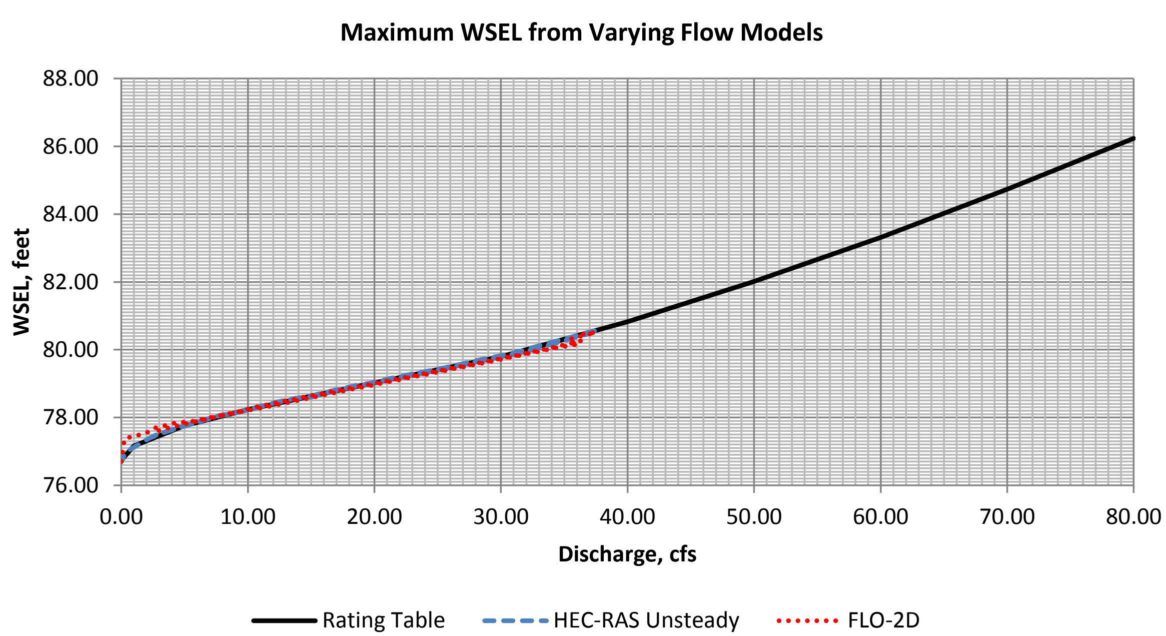


Figure 4.87 Scenario 8 Varying Flow Hydrograph Results

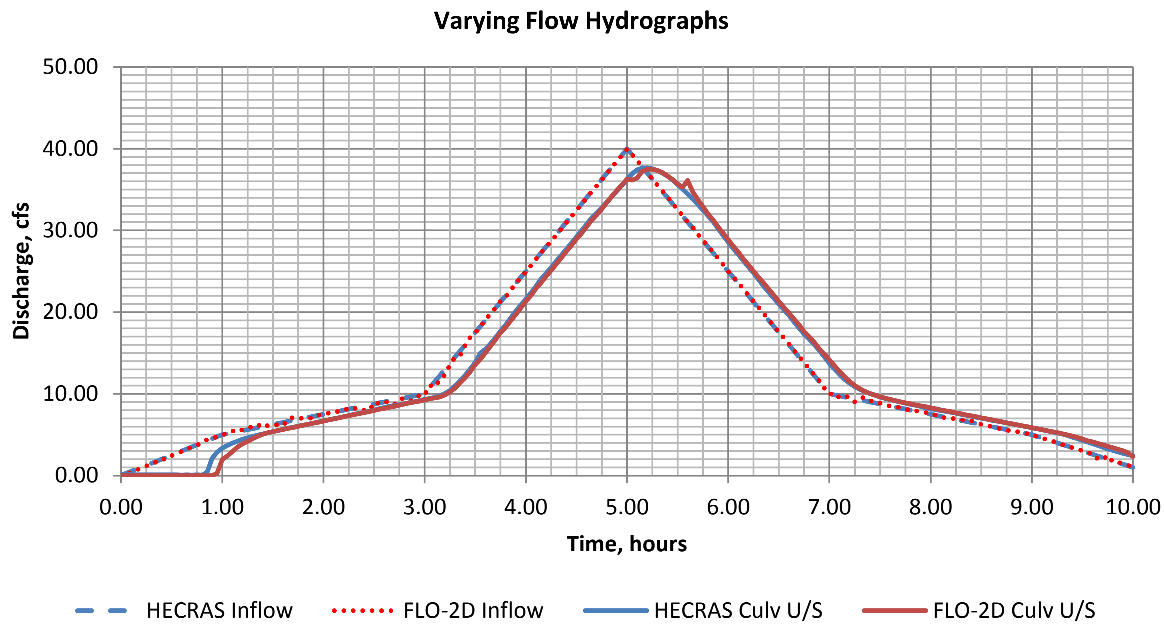


Figure 4.88 Scenario 8 Uniform Flow Hydrograph Results

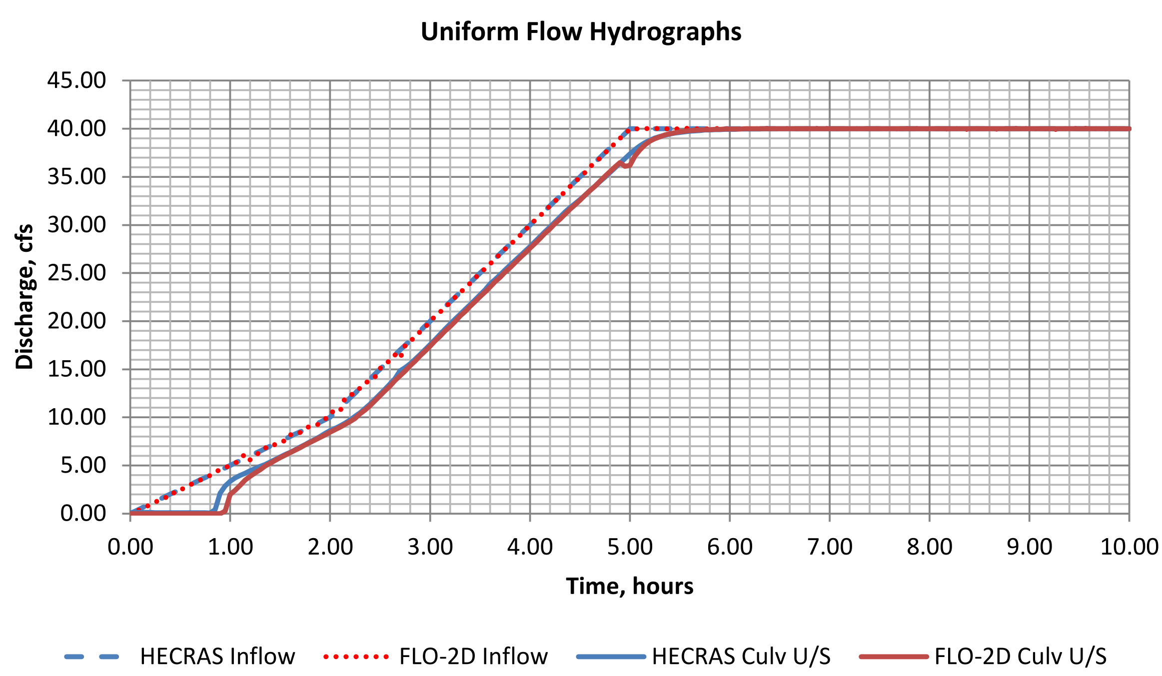


Figure 4.89 Scenario 8 Varying Flow *WSEL* Profile Results

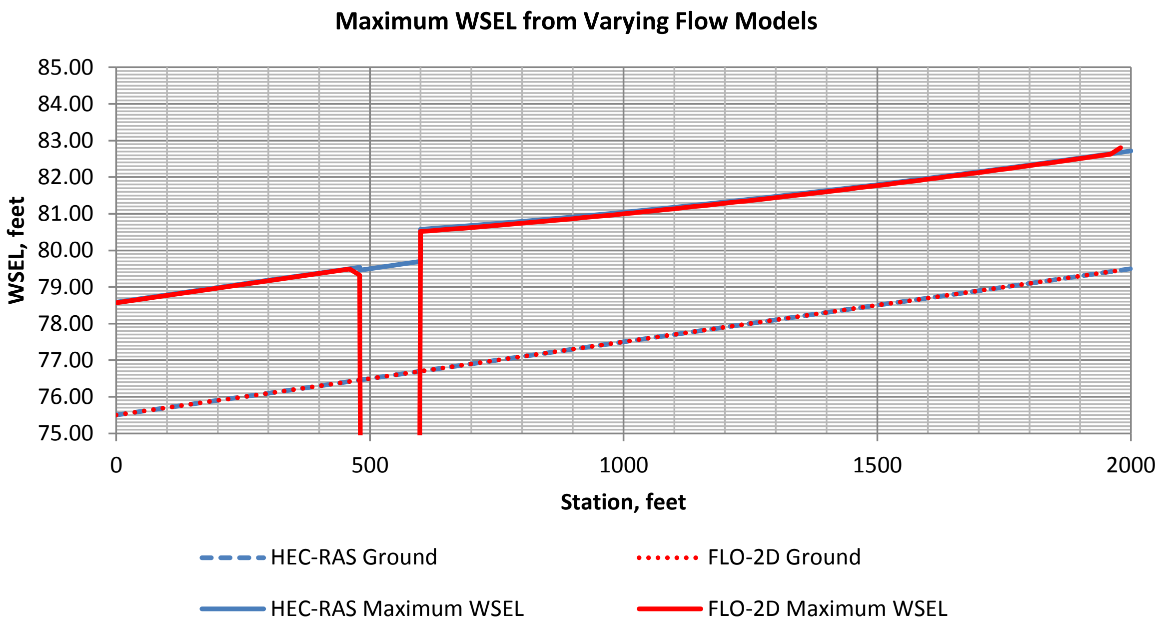


Figure 4.90 Scenario 8 Uniform Flow *WSEL* Profile Results

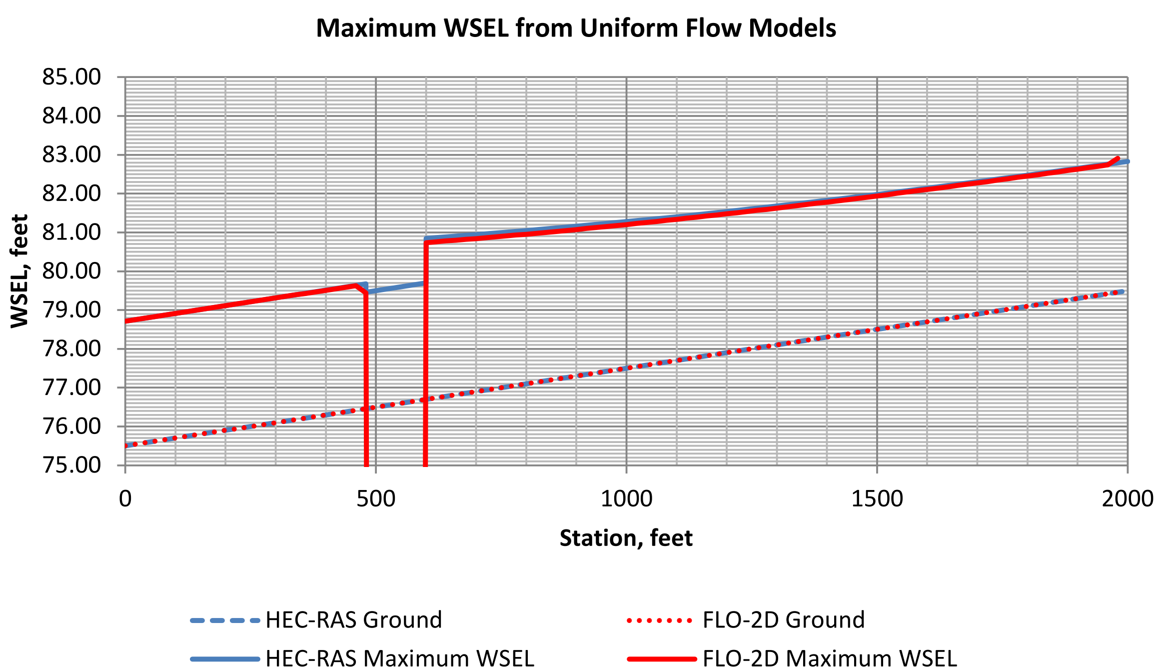
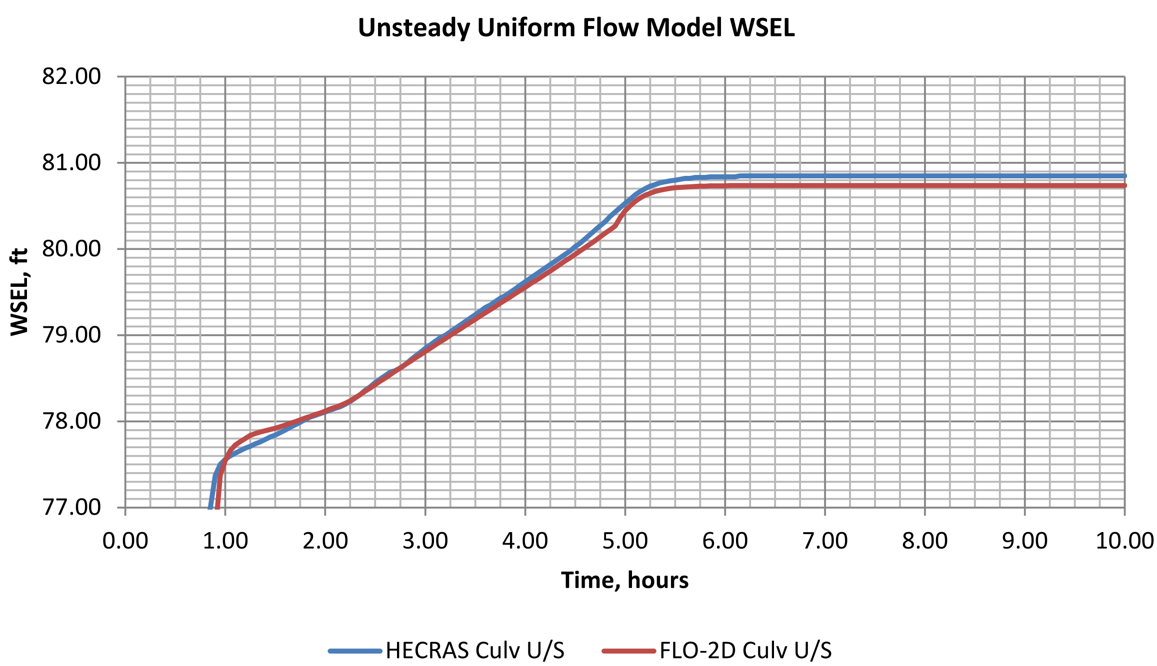


Figure 4.91 Scenario 8 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.10 Scenario 9: Rating Table Channel to Channel (IC and OC)

Scenario

Test of the *FLO-2D* rating table approach for a culvert connecting two *1D* channel grids, switching between inlet and outlet control during the model run.

Results and Discussion

Scenario 9 is the same as Scenario 7 except that the slope was changed to 0.4500% to force an inlet/outlet control condition. The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.29. The *FLO-2D* results for the culvert at peak flow are very close to unsteady *HEC-RAS*.

The results from Benchmark 1 are shown on Figure 4.92. The rating table option based results are very stable and are almost identical to the rating table generated using steady state *HEC-RAS* and the unsteady *HEC-RAS* results.

The results from Benchmark 2 are shown on Figure 4.93 and Figure 4.94. The *FLO-2D* results are very stable and compare very well with unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.93 and Figure 4.94. The *RMSD* for the varying flow culvert inlet hydrograph was 0.44 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.95 and Figure 4.96. The *WSEL* profiles downstream and upstream of the culvert for the varying and uniform flow models match *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.95 and Figure 4.96. The *RMSD* for the varying maximum *WSEL* profile was 0.02 feet, which is a very minor deviation.

The results from Benchmark 4 are shown on Figure 4.97. The *FLO-2D WSEL* matches unsteady *HEC-RAS* very well for the uniform flow case at the inlet, being only 0.05 feet lower.

The *FLO-2D* results for Scenario 9, as compared with the unsteady *HEC-RAS* results, are acceptable for *FCDMC* purposes.

<table>
<tr><th colspan="5">Table 4.29 Scenario 9 Comparison of Hydraulic Structure Model Results</th></tr>
<tr><th rowspan="2">Model</th><th colspan="2">Water Surface Elevation at Peak</th><th rowspan="2">Discharge through Culvert, cfs</th><th rowspan="2">Percent Difference</th></tr>
<tr><th>Inlet</th><th>Outlet</th></tr>
<tr><td>FLO-2D Varying</td><td>83.20</td><td>80.84</td><td>57.66</td><td rowspan="2">0.10</td></tr>
<tr><td>HEC-RAS Varying</td><td>83.24</td><td>80.79</td><td>57.60</td></tr>
<tr><td>FLO-2D Uniform</td><td>83.49</td><td>80.93</td><td>60.00</td><td rowspan="2">0.00</td></tr>
<tr><td>HEC-RAS Uniform</td><td>83.54</td><td>80.95</td><td>60.00</td></tr>
</table>

Figure 4.92 Scenario 9 Rating Table Floodplain to Floodplain (OC) Test Results

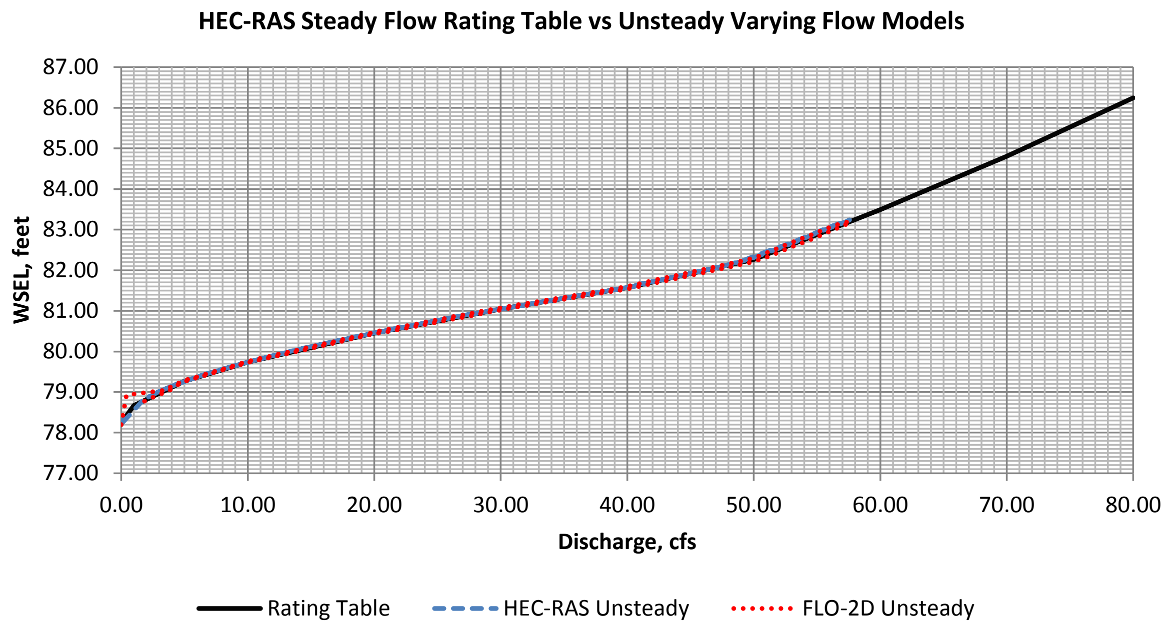


Figure 4.93 Scenario 9 Varying Flow Hydrograph Results

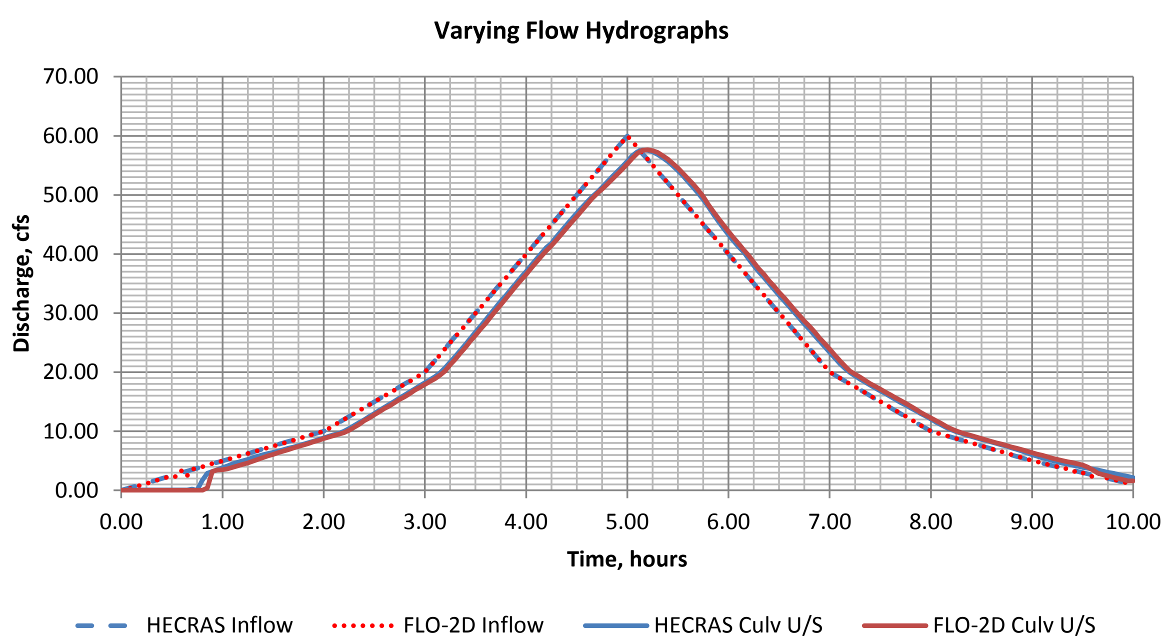


Figure 4.94 Scenario 9 Uniform Flow Hydrograph Results

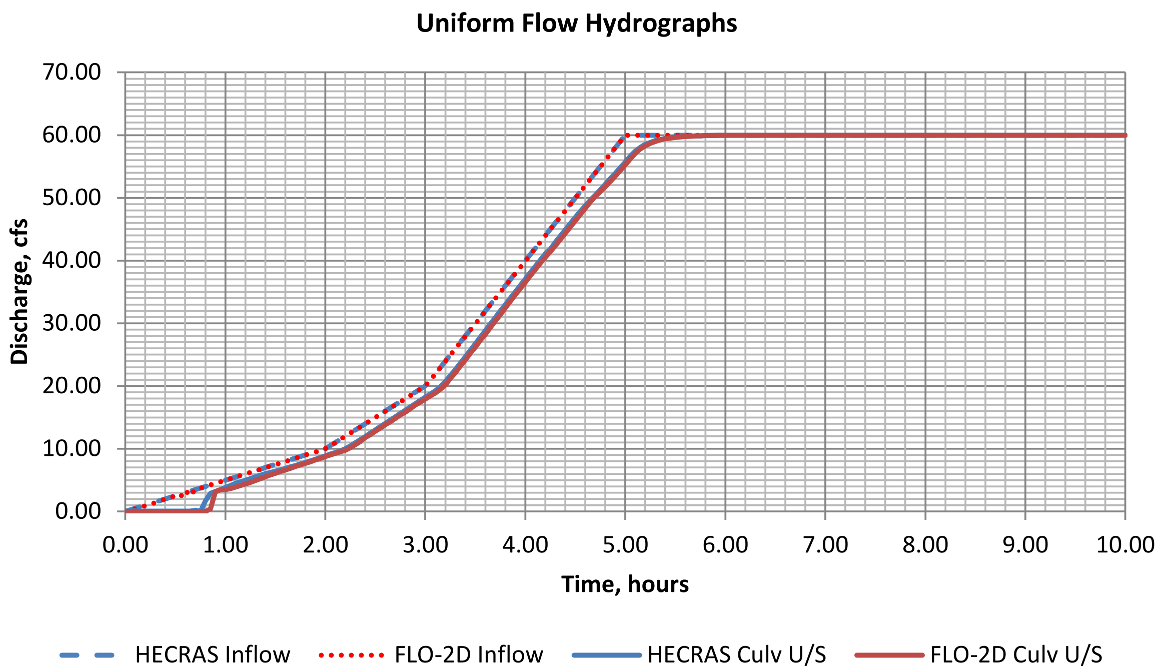


Figure 4.95 Scenario 9 Varying Flow *WSEL* Profile Results

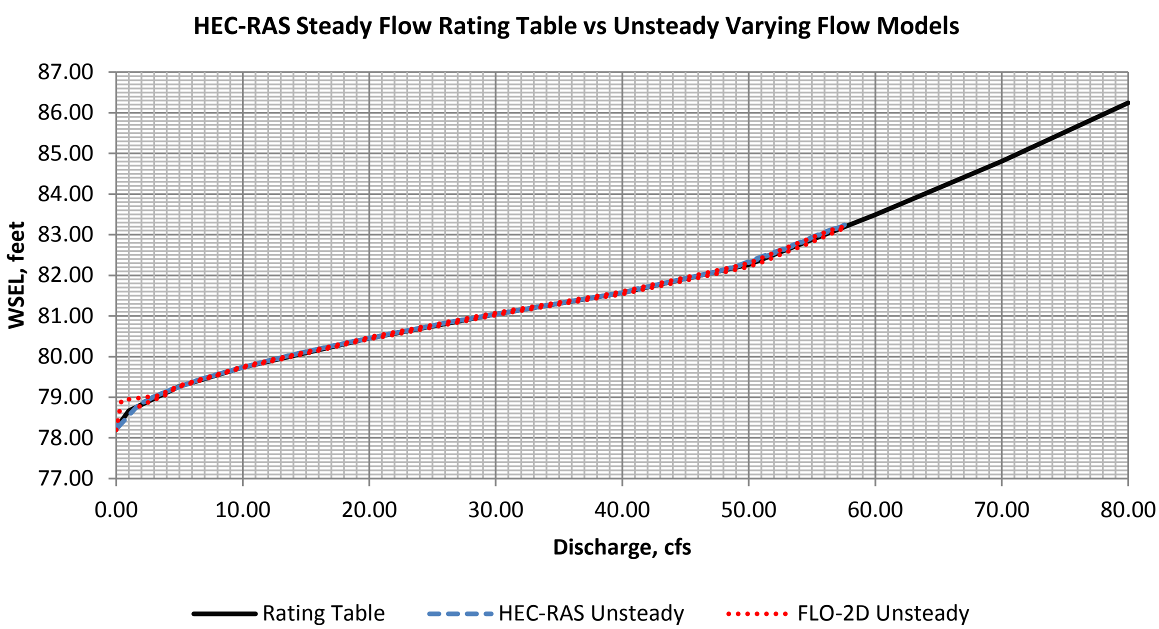


Figure 4.96 Scenario 9 Uniform Flow *WSEL* Profile Results

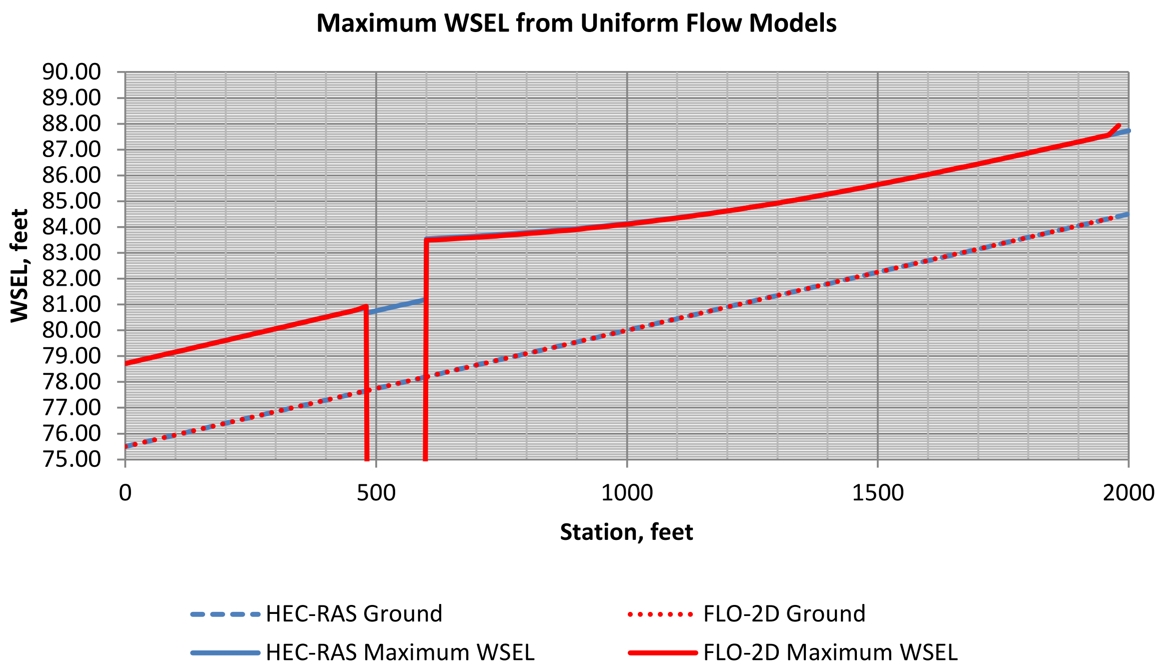
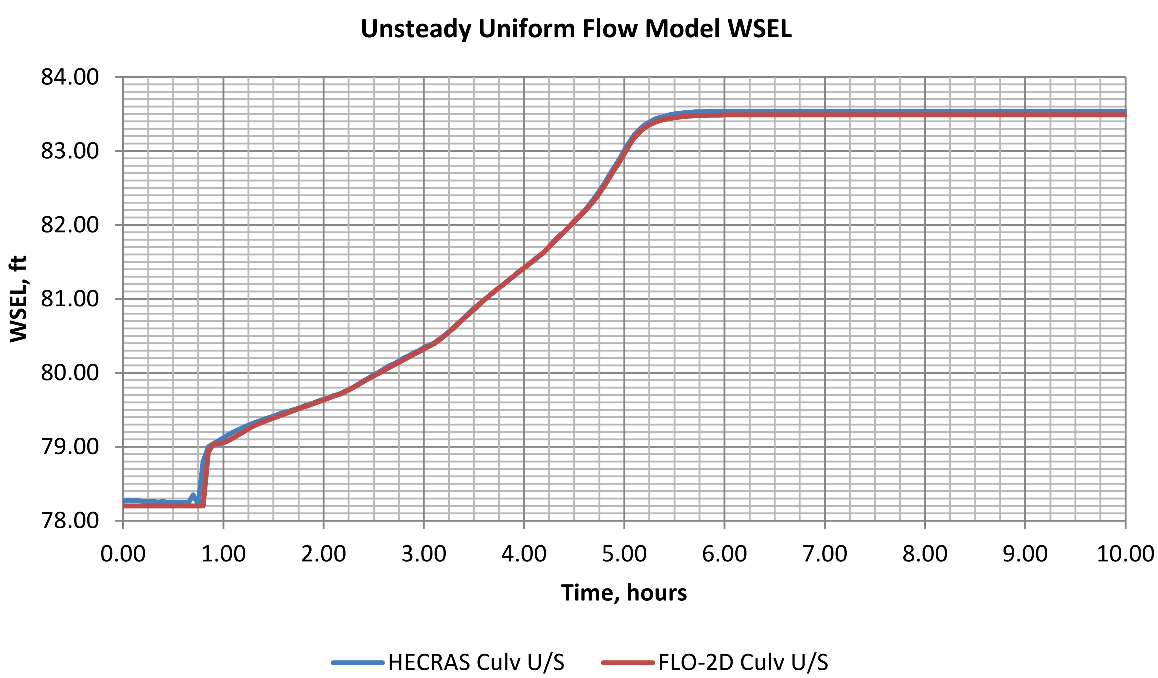


Figure 4.97 Scenario 9 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.11 Scenario 10: General Equations Channel to Channel (IC and OC)

Scenario

Test of the *FLO-2D* general culvert equations approach for a culvert connecting two *1D* channel elements, switching between inlet and outlet control during the model run.

Results and Discussion

Scenario 10 is the same as Scenario 9 except that the general culvert equations option was used instead of the rating table option**.** The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.30. The *FLO-2D* results for the culvert at peak flow are significantly different than unsteady *HEC-RAS,* 0.18 to 0.27 feet lower at the inlet, and 0.36 to 0.41 feet lower at the outlet. The differences are due to an algorithm difference between *FLO-2D* and *HEC-RAS* at the outlet for this particular hydraulic condition. It carries through to the inlet. A check of channel hydraulics based on depth-variable Manning's *n* is shown in Table 4.31 for the uniform flow hydrograph condition at Station 2+00 (Grid 611). The *FLO-2D* results are identical to unsteady *HEC-RAS.*

The results from Benchmark 1 are shown on Figure 4.98. The general culvert equation-based results are a lower than *HEC-RAS* in the range of 50 to 60 cfs; otherwise, they are closely match the rating table generated using steady state *HEC-RAS* and the unsteady *HEC-RAS* results.

The results from Benchmark 2 are shown on Figure 4.99 and Figure 4.100. The *FLO-2D* results are very stable and compare very well with unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.51 and Figure 4.52. The *RMSD* for the varying flow culvert inlet hydrograph was 0.46 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.101 and Figure 4.102. The *WSEL* profiles downstream and upstream of the culvert for the varying and uniform flow models match *HEC-RAS* very well. However, the maximum *WSEL* at the inlet and outlet are lower than *HEC-RAS* as noted above. The lower *WSEL* at the outlet, which is also lower than the immediate downstream profile, appears to be controlling the lower *WSEL* at the inlet. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.46 and Figure 4.47. The *RMSD* for the varying maximum *WSEL* profile was 0.08 feet, which is a minor deviation.

The results from Benchmark 4 are shown on Figure 4.103. The *FLO-2D WSEL* matches unsteady *HEC-RAS* very closely, except near and at peak, where the *FLO-2D WSEL* is about 0.18 to 0.27 feet lower than unsteady *HEC-RAS*, as also noted above.

The differences between the *FLO-2D* and unsteady *HEC-RAS* results for Scenario 10 are acceptable for *FCDMC* purposes. However, the rating table approach may provide better results depending on the physical conditions at an actual site (refer to Scenario 9). Refer to Section 7.8 for guidance.

Table 4.30 Scenario 10 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	83.06	80.38	58.18	1.01
HEC-RAS Varying	83.24	80.79	57.60	
FLO-2D Uniform	83.27	80.51	60.00	0.00
HEC-RAS Uniform	83.54	80.87	60.00	

Table 4.31 Scenario 10 Comparison of Depth and Velocity at Station 2+00 (Grid 611)

***FLO-2D* at Time 5.55 hours**					***HEC-RAS***				
Depth, ft	**Area, sf**	**Wetted Perimeter, ft**	**n**	**Velocity, fps**	**Depth, ft**	**Area, sf**	**Wetted Perimeter, ft**	**n**	**Velocity, fps**
3.21	26.60	14.70	0.066	2.26	3.21	26.63	14.70	0.066	2.26
Computed by *FLO-2D*:				2.26	Computed by *HEC-RAS*:				2.25
Discharge by *FLO-2D*:				60.00	Discharge:				60.00
	Computed using Manning's equation								

Figure 4.98 Scenario 10 Rating Table Floodplain to Floodplain (OC) Test Results

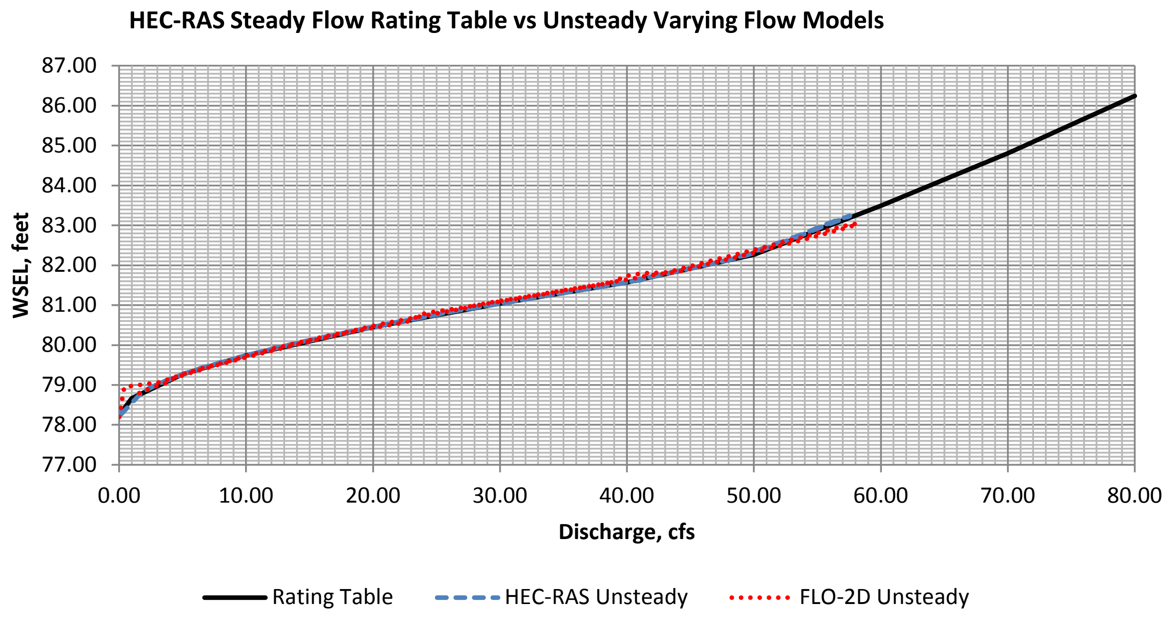


Figure 4.99 Scenario 10 Varying Flow Hydrograph Results

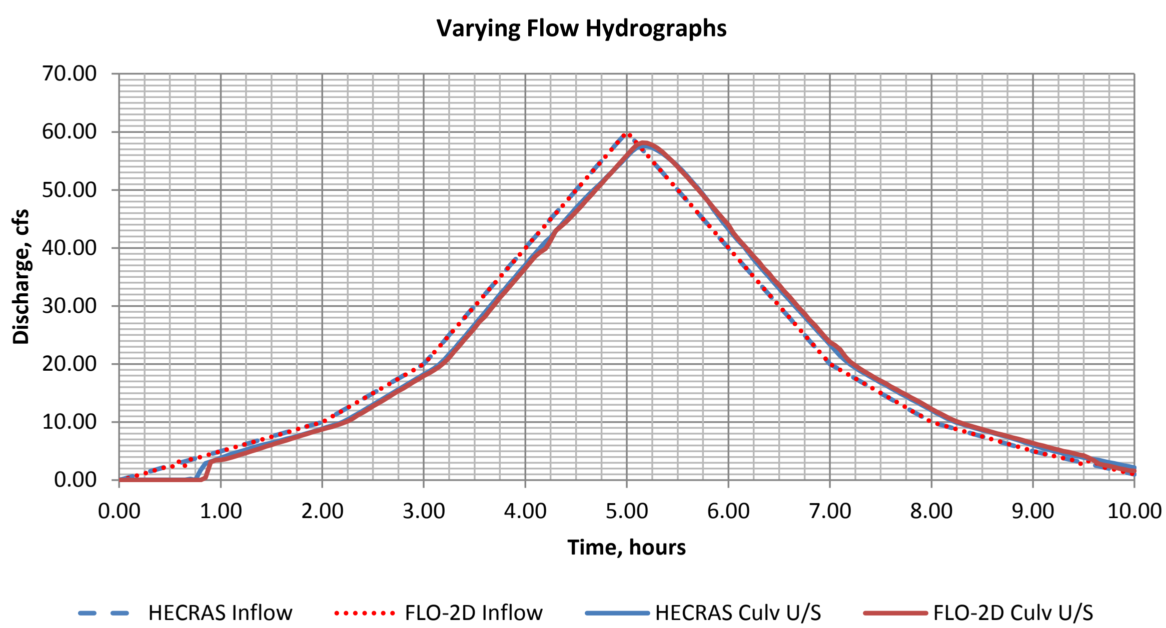


Figure 4.100 Scenario 10 Uniform Flow Hydrograph Results

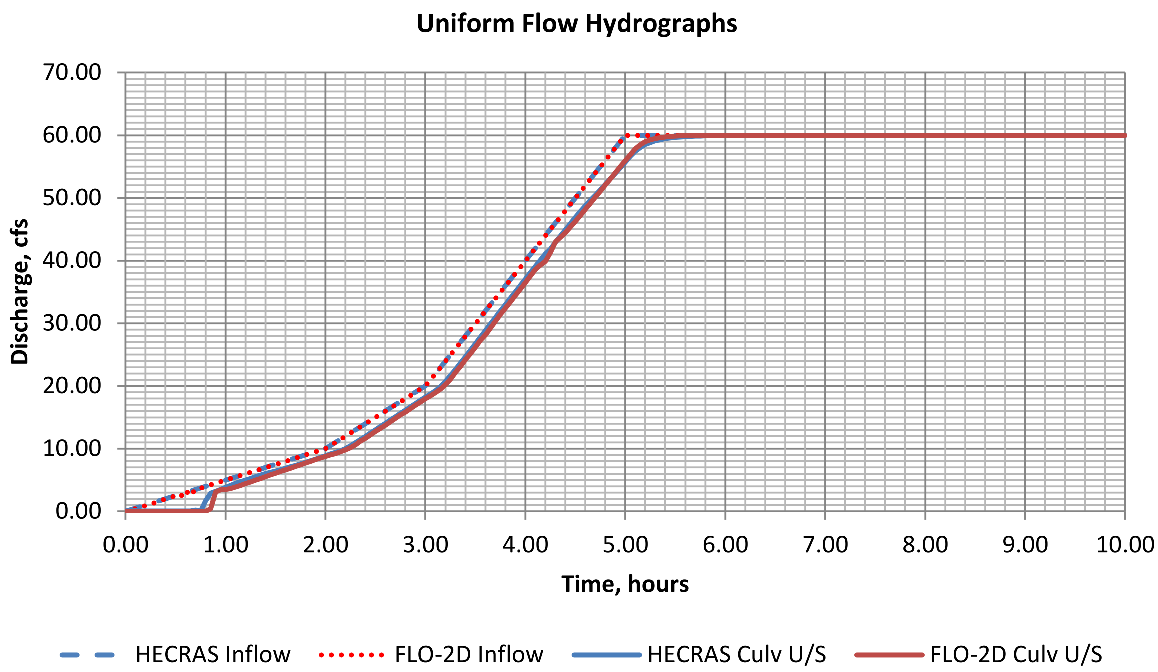


Figure 4.101 Scenario 10 Varying Flow *WSEL* Profile Results

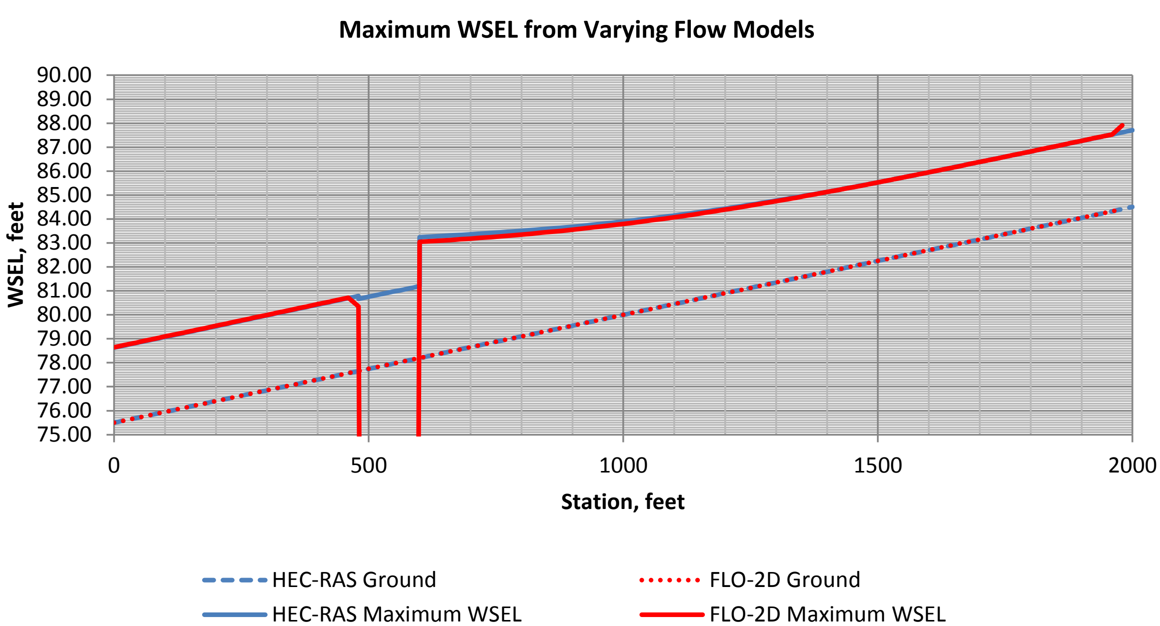


Figure 4.102 Scenario 10 Uniform Flow *WSEL* Profile Results

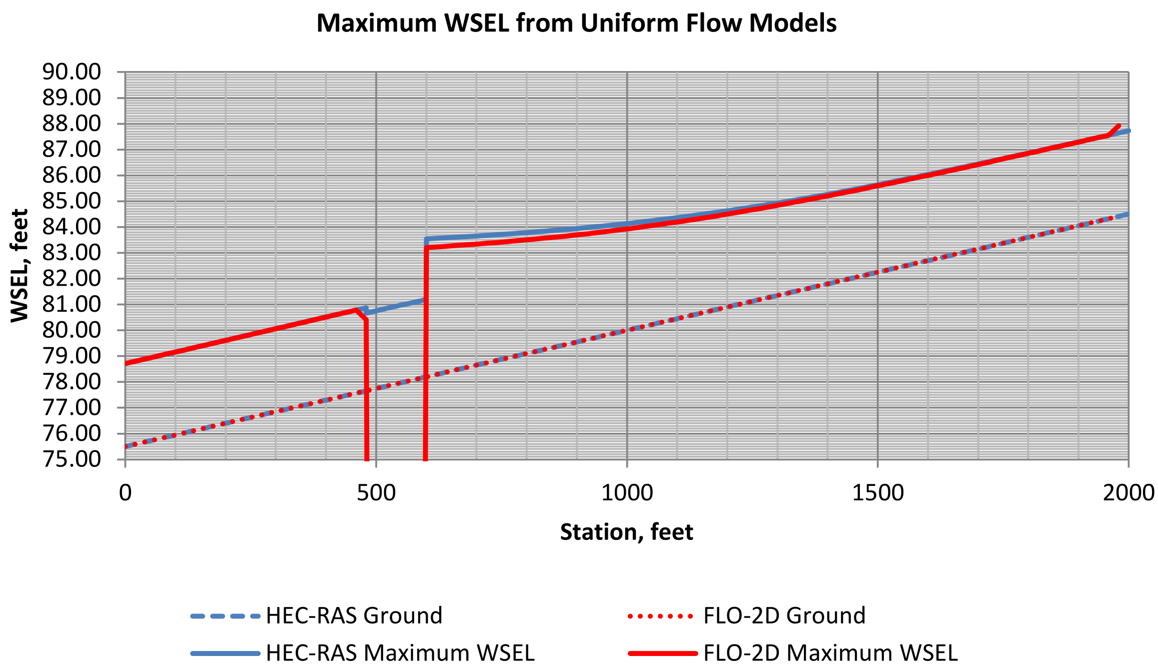
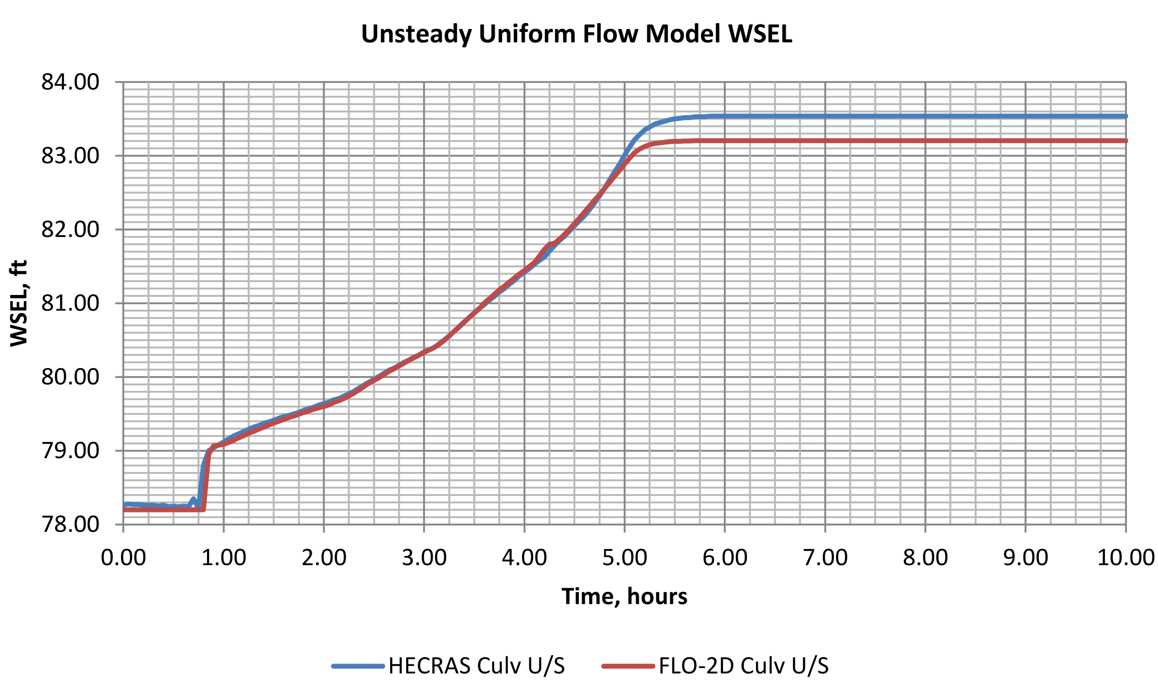


Figure 4.103 Scenario 10 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.12 Scenario 11: General Equations Channel to Channel (IC and OC) – Backwater

Under development. To be provided in a future update. This scenario is not important for *FEMA* approval.

4.2.5.13 Scenario 12: Rating Table Floodplain to Channel (OC)

Scenario

Test of the *FLO-2D* rating table approach for a culvert connecting upstream floodplain grids to a downstream *1D* channel, operating under outlet control.

Results and Discussion

Scenario 12 is similar to Scenario 7 except that floodplain grids were used to form the upstream channel, and the *1D* channel option was used for the reach downstream from the culvert. The culvert was assigned to connect a floodplain grid to a *1D* channel grid. The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.32. The *FLO-2D* results for the culvert at peak flow are comparable with unsteady *HEC-RAS*, but there is more numerical instability than the Scenario 7 models resulting in a larger difference in maximum *WSEL* and discharge at the culvert inlet and outlet. The difference in peak discharge of 6% is less than the 10% difference allowance set forth previously and is acceptable for *FCDMC* purposes.

The results from Benchmark 1 are shown on Figure 4.104. The rating table-based results are a little unstable near peak but otherwise are almost identical to the rating table generated using steady state *HEC-RAS* and the unsteady *HEC-RAS* results.

The results from Benchmark 2 are shown on Figure 4.105 and Figure 4.106. The *FLO-2D* results are unstable near the peak discharge but otherwise compare very well with unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.51 and Figure 4.52. The *RMSD* for the varying flow culvert inlet hydrograph was 0.59 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.107 and Figure 4.108. The *WSEL* profiles downstream of the culvert for the varying and uniform flow models match *HEC-RAS* very well. Upstream, the results are comparable with unsteady *HEC-RAS* but are slightly unstable and tend to be a little higher than *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.46 and

Figure 4.47. The *RMSD* for the varying maximum *WSEL* profile was 0.12 feet, which is a minor deviation.

The results from Benchmark 4 are shown on Figure 4.109. The results are unsteady approaching and at peak but trend to match unsteady *HEC-RAS* fairly well.

The minor differences between the *FLO-2D* and unsteady *HEC-RAS* results for Scenario 12 are acceptable for *FCDMC* purposes.

Table 4.32 Scenario 12 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	80.49	79.58	38.64	2.74
HEC-RAS Varying	80.57	79.54	37.61	
FLO-2D Uniform	80.68	79.79	42.45	6.13
HEC-RAS Uniform	80.85	79.67	40.00	

Figure 4.104 Scenario 12 Rating Table Floodplain to Floodplain (OC) Test Results

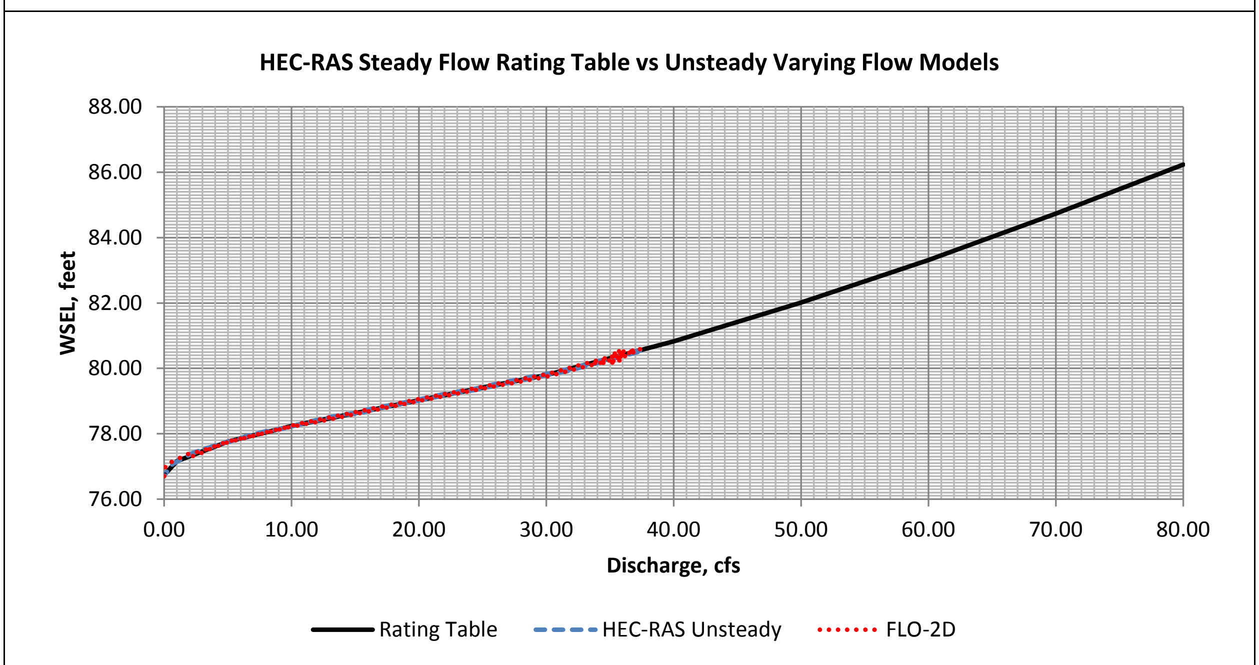


Figure 4.105 Scenario 12 Varying Flow Hydrograph Results

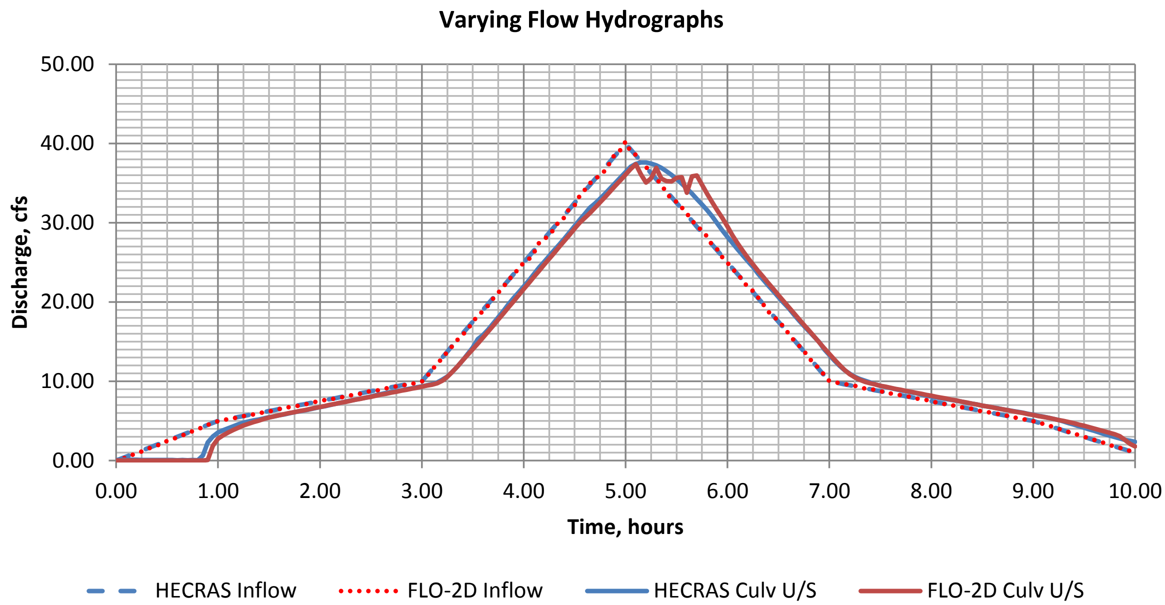


Figure 4.106 Scenario 12 Uniform Flow Hydrograph Results

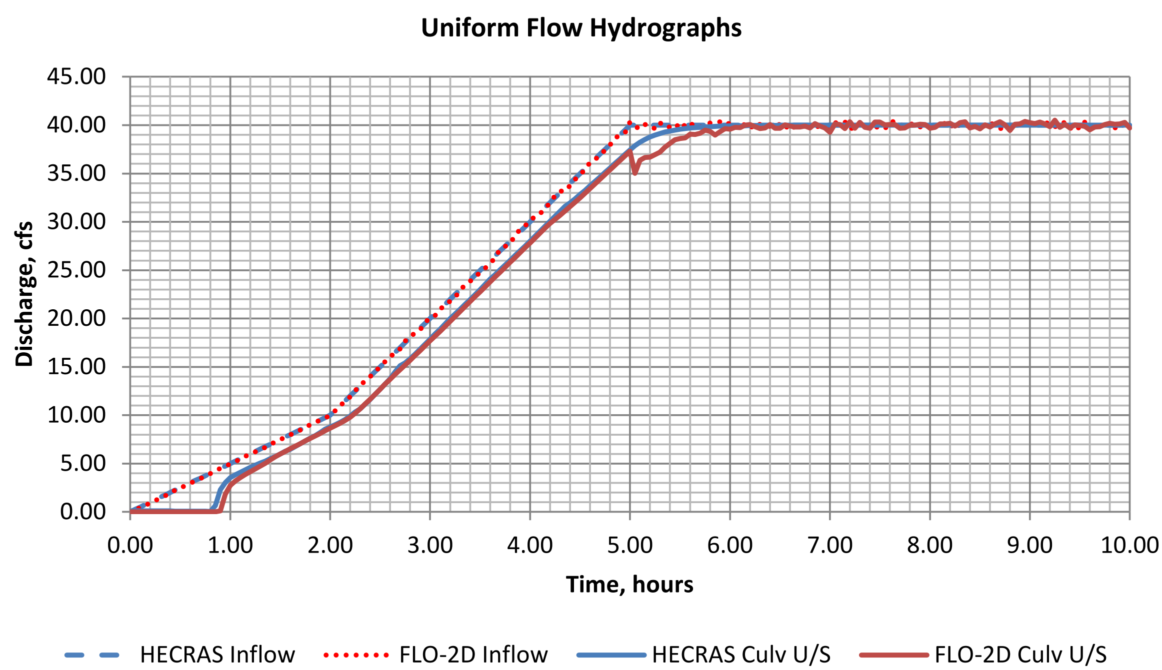


Figure 4.107 Scenario 12 Varying Flow *WSEL* Profile Results

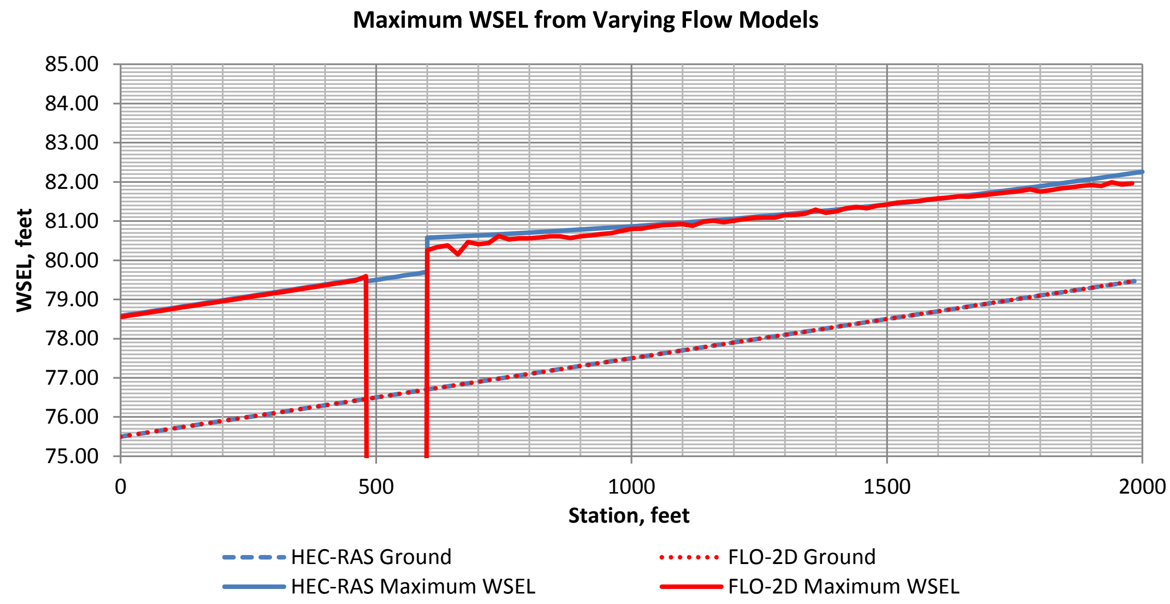


Figure 4.108 Scenario 12 Uniform Flow *WSEL* Profile Results

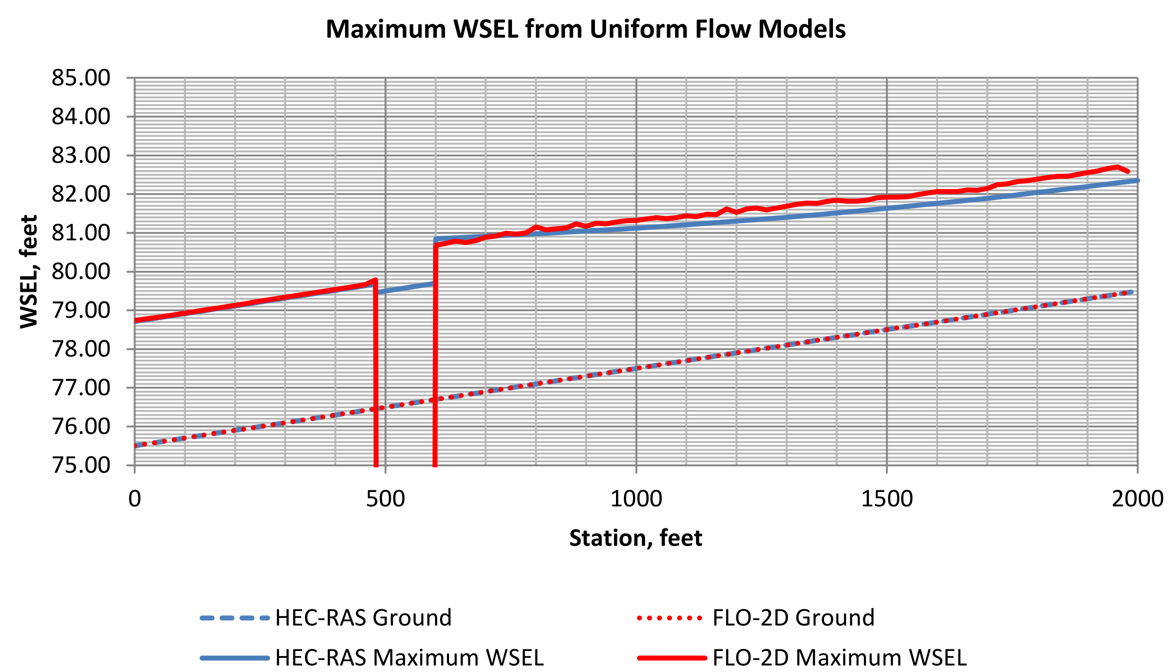
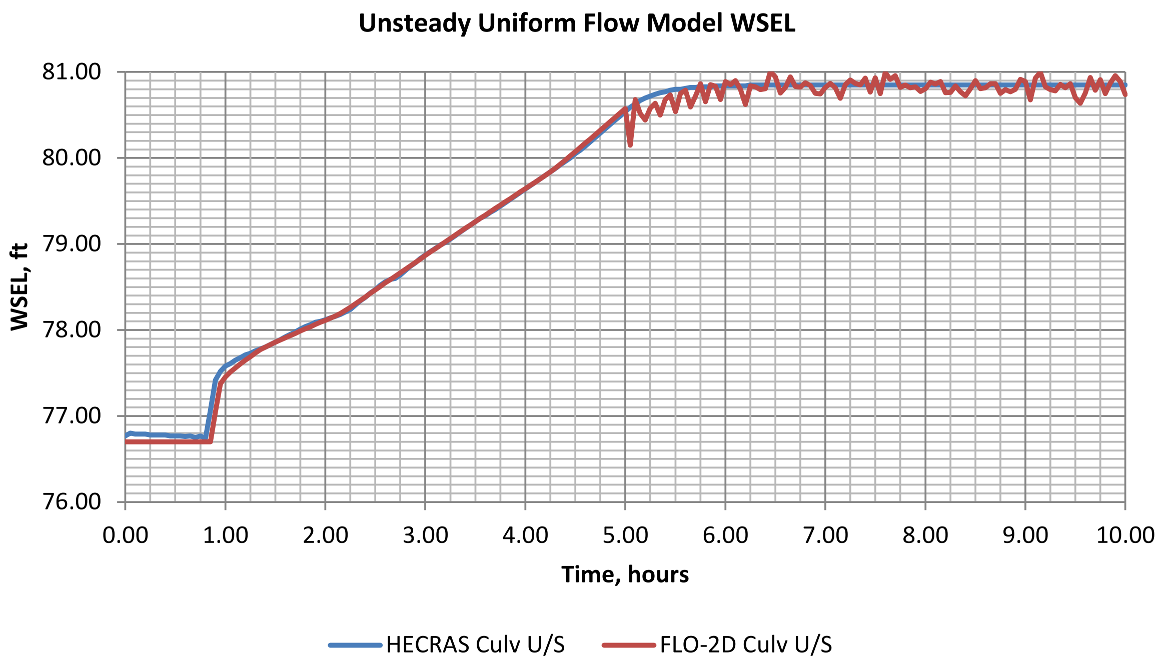


Figure 4.109 Scenario 12 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.14 Scenario 13: General Equations Floodplain to Channel (OC)

Scenario

Test of the *FLO-2D* general culvert equations approach for a culvert connecting upstream floodplain grids to a downstream *1D* channel, operating under outlet control.

Results and Discussion

Scenario 13 is the similar to Scenario 12 except that the general culvert equations option is used instead of the rating table approach. The culvert was assigned to connect a floodplain grid to a *1D* channel grid. The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.33. The *FLO-2D* results for the culvert at peak flow are not as good a match with unsteady *HEC-RAS* because there is significant numerical instability approaching and at peak discharge, resulting in a larger difference in maximum *WSEL* at the culvert inlet and outlet than was seen in Scenarios 7 and 12.

The results from Benchmark 1 are shown on Figure 4.110. The rating table-based results are very unstable near peak but otherwise are almost identical to the rating table generated using steady state *HEC-*

RAS and the unsteady *HEC-RAS* results. The instability in discharge near the time to peak without any appreciable corresponding fluctuation in *WSEL* is what causes the fluctuations noted.

The results from Benchmark 2 are shown on Figure 4.111 and Figure 4.112. The *FLO-2D* results are very unstable near the peak discharge. However, they are nearly identical to the rating table generated using steady-state *HEC-RAS* and the unsteady *HEC-RAS* model results. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.111 and Figure 4.112. The *RMSD* for the varying flow culvert inlet hydrograph was 0.88 cfs, which is a minor deviation.

The results from Benchmark 3 are shown on Figure 4.113 and Figure 4.114. The *WSEL* profiles downstream of the culvert for the varying and uniform flow models match *HEC-RAS* fairly well except at the culvert inlet and outlet for the varying flow model. Upstream, the results are comparable with unsteady *HEC-RAS* but are slightly unstable and tend to be a little higher than *HEC-RAS*, particularly for the uniform flow model. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.113 and Figure 4.114. The *RMSD* for the varying maximum *WSEL* profile was 0.24 feet, which is relatively minor deviation primarily due to numeric instability in *FLO-2D* for this test case.

The results from Benchmark 4 are shown on Figure 4.115. The results are unstable approaching and at peak but trend to match unsteady *HEC-RAS* fairly well.

The differences between the *FLO-2D* and unsteady *HEC-RAS* results for Scenario 13 are generally acceptable for *FCDMC* planning purposes, but a lower Courant or application of the Rating Table method may be necessary. The user should carefully examine the model results when applying the general culvert equations for similar hydraulic conditions.

Table 4.33 Scenario 13 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	80.20	79.81	44.46	18.21
HEC-RAS Varying	80.57	79.54	37.61	
FLO-2D Uniform	80.76	79.82	44.39	10.98
HEC-RAS Uniform	80.85	79.67	40.00	

Figure 4.110 Scenario 13 Rating Table Floodplain to Floodplain (OC) Test Results

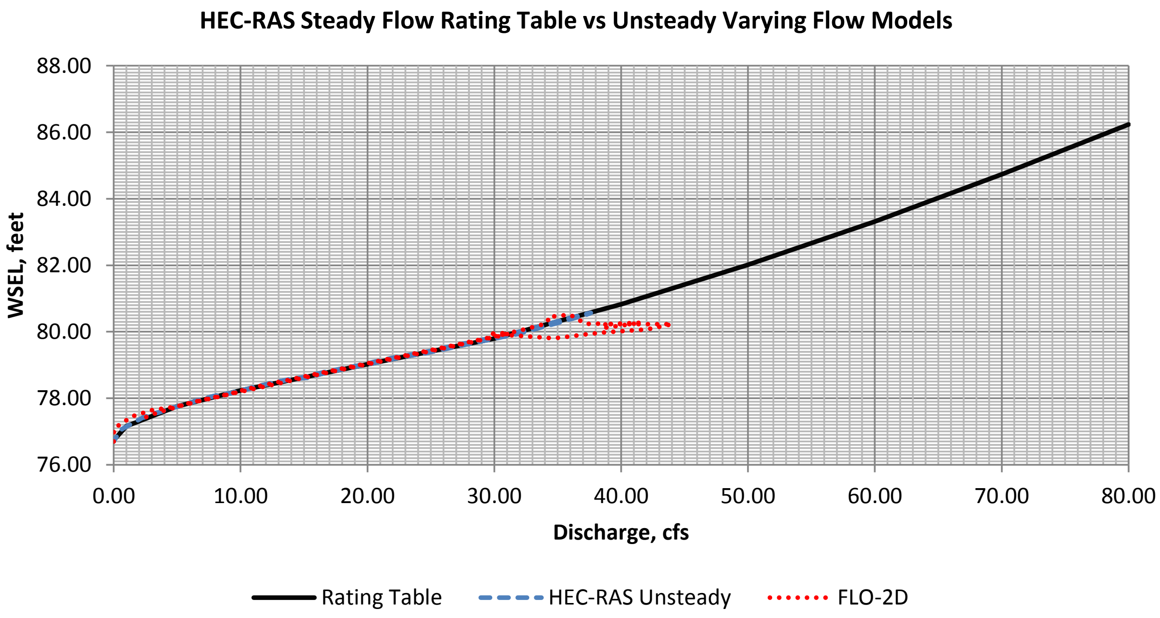


Figure 4.111 Scenario 13 Varying Flow Hydrograph Results

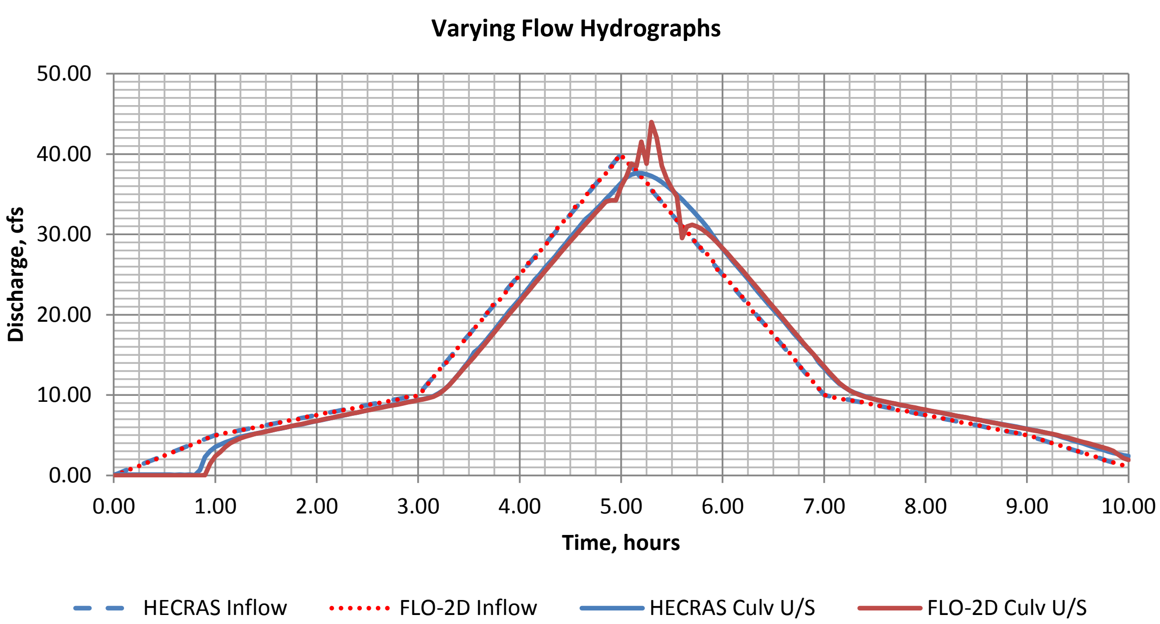


Figure 4.112 Scenario 13 Uniform Flow Hydrograph Results

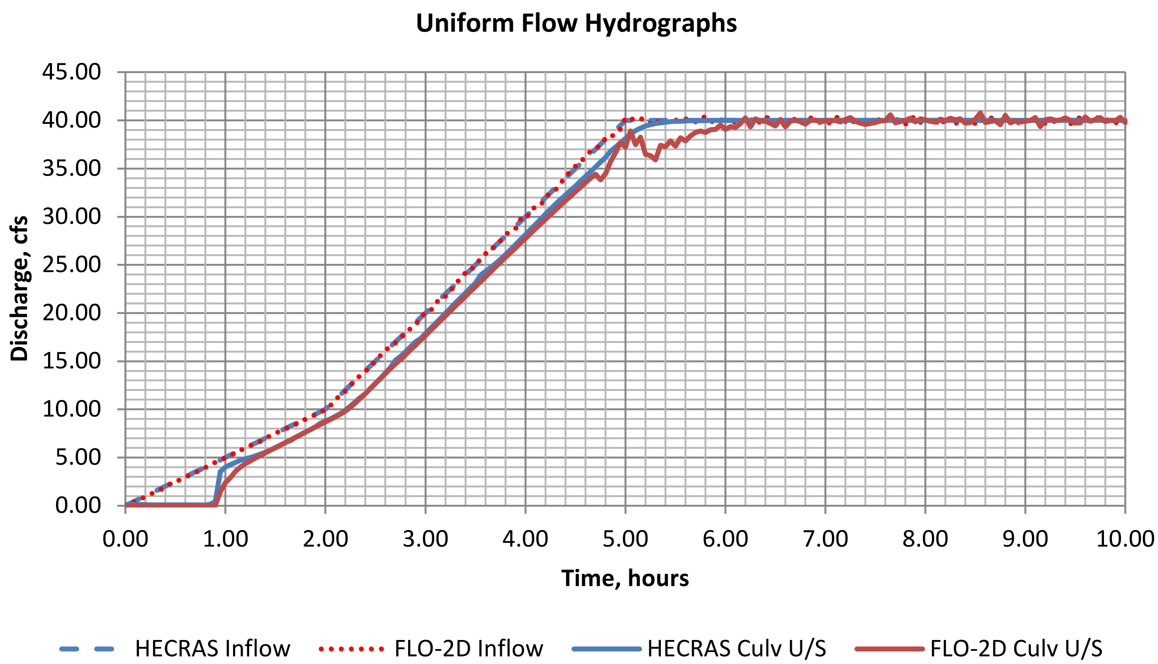


Figure 4.113 Scenario 13 Varying Flow *WSEL* Profile Results

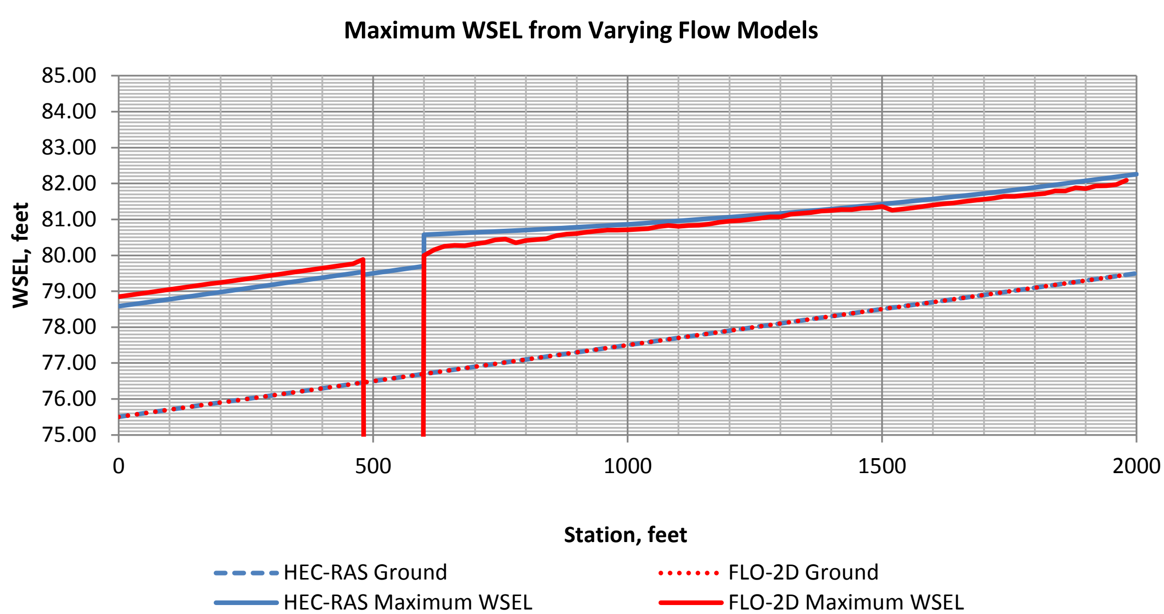


Figure 4.114 Scenario 13 Uniform Flow *WSEL* Profile Results

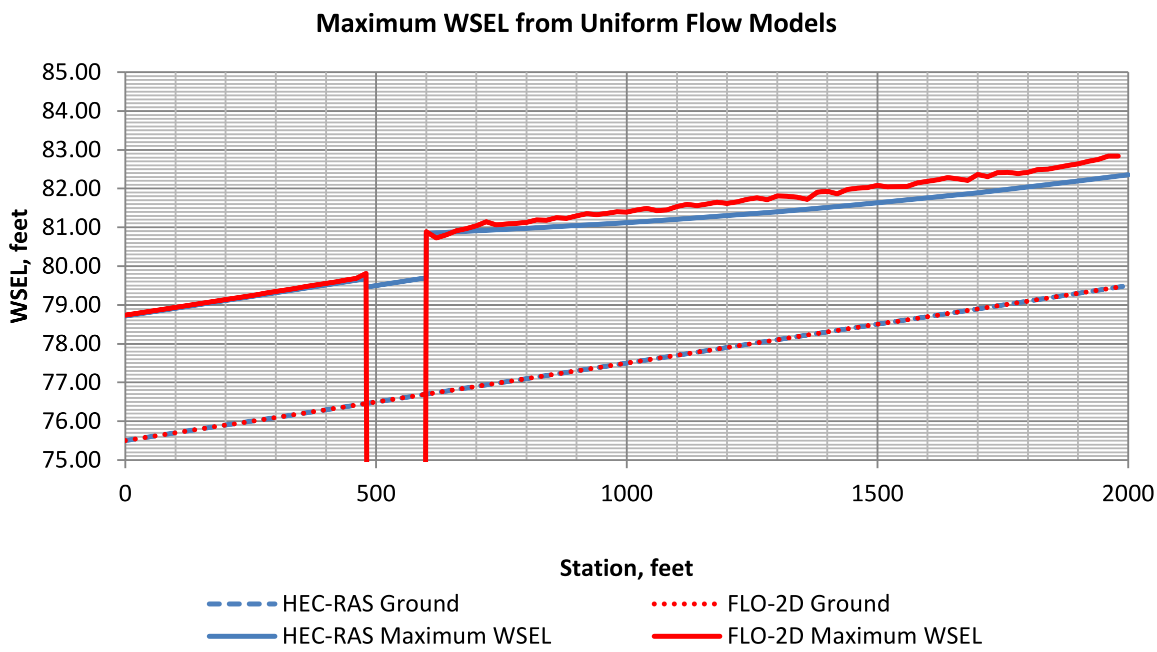
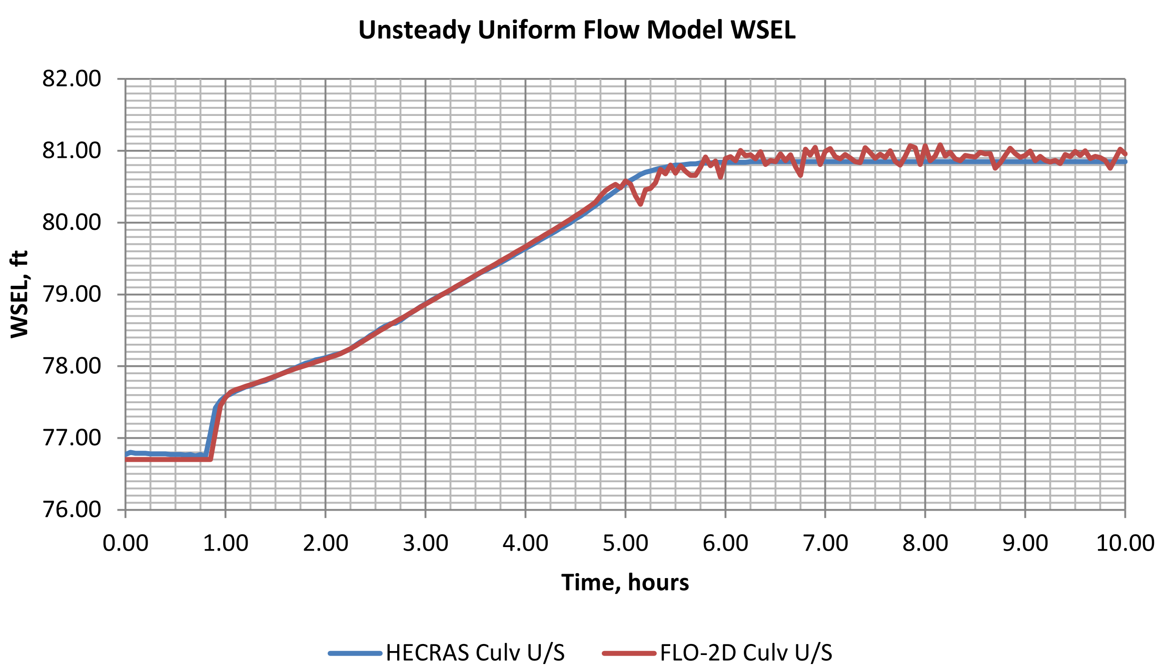


Figure 4.115 Scenario 13 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.15 Scenario 14: Rating Table Channel to Floodplain (OC)

Scenario

Test of the *FLO-2D* rating table approach for a culvert connecting an upstream *1D* channel to downstream floodplain grids, operating under outlet control.

Results and Discussion

Scenario 14 is similar to Scenario 12 except that floodplain grids were used to form the downstream channel, and the *1D* channel option was used for the reach upstream from the culvert. The culvert was assigned to connect a *1D* channel grid to a floodplain grid. The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.34. The *FLO-2D* results for the culvert at peak flow are almost identical with unsteady *HEC-RAS*.

The results from Benchmark 1 are shown on Figure 4.116. The rating table-based results are almost identical to the rating table generated using steady state *HEC-RAS* and the unsteady *HEC-RAS* results.

The results from Benchmark 2 are shown on Figure 4.117 and Figure 4.118. The *FLO-2D* results are also nearly identical with unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.117 and Figure 4.118. The *RMSD* for the varying flow culvert inlet hydrograph was 0.36 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.119 and Figure 4.120. The *WSEL* profiles downstream of the culvert for the varying and uniform flow models are approximately 0.1 feet lower then *HEC-RAS*, as expected due to the difference in wetted perimeter. Upstream, the results are nearly identical to unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.119 and Figure 4.120. The *RMSD* for the varying maximum *WSEL* profile was 0.05 feet, which is a very minor deviation.

The results from Benchmark 4 are shown on Figure 4.121. The results match unsteady *HEC-RAS* very well.

The very minor differences between the *FLO-2D* and unsteady *HEC-RAS* results for Scenario 12 are acceptable for *FCDMC* purposes.

Table 4.34 Scenario 14 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	80.22	79.03	38.45	0.26
HEC-RAS Varying	80.23	79.12	38.35	
FLO-2D Uniform	80.33	79.09	40.00	0.00
HEC-RAS Uniform	80.36	79.18	40.00	

Figure 4.116 Scenario 14 Rating Table Floodplain to Floodplain (OC) Test Results

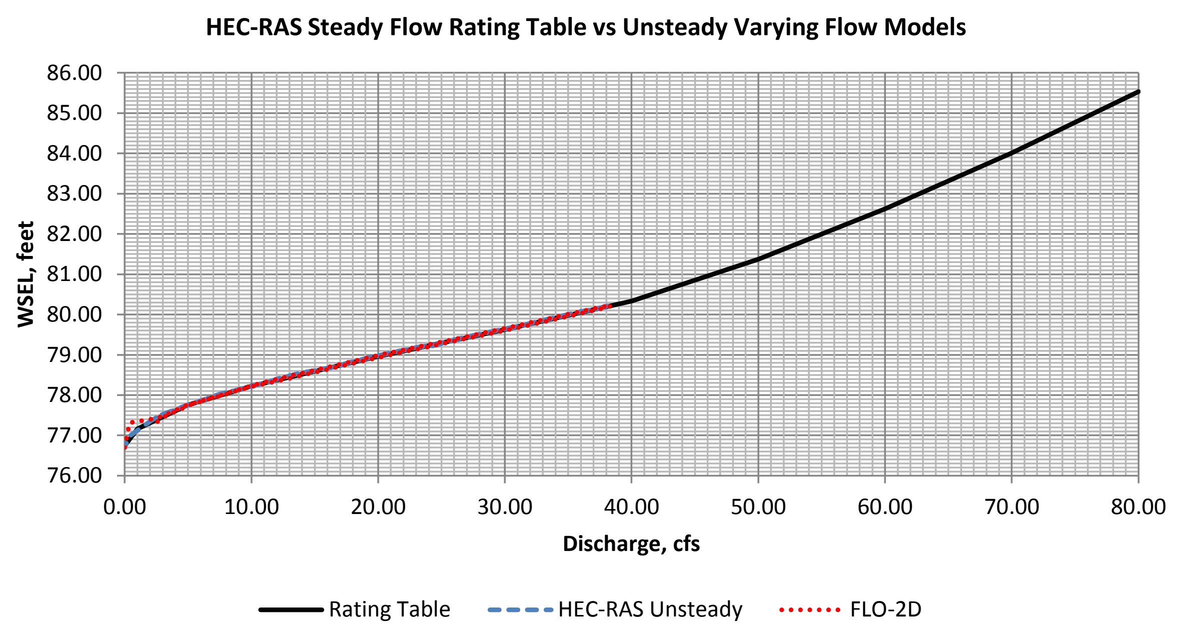


Figure 4.117 Scenario 14 Varying Flow Hydrograph Results

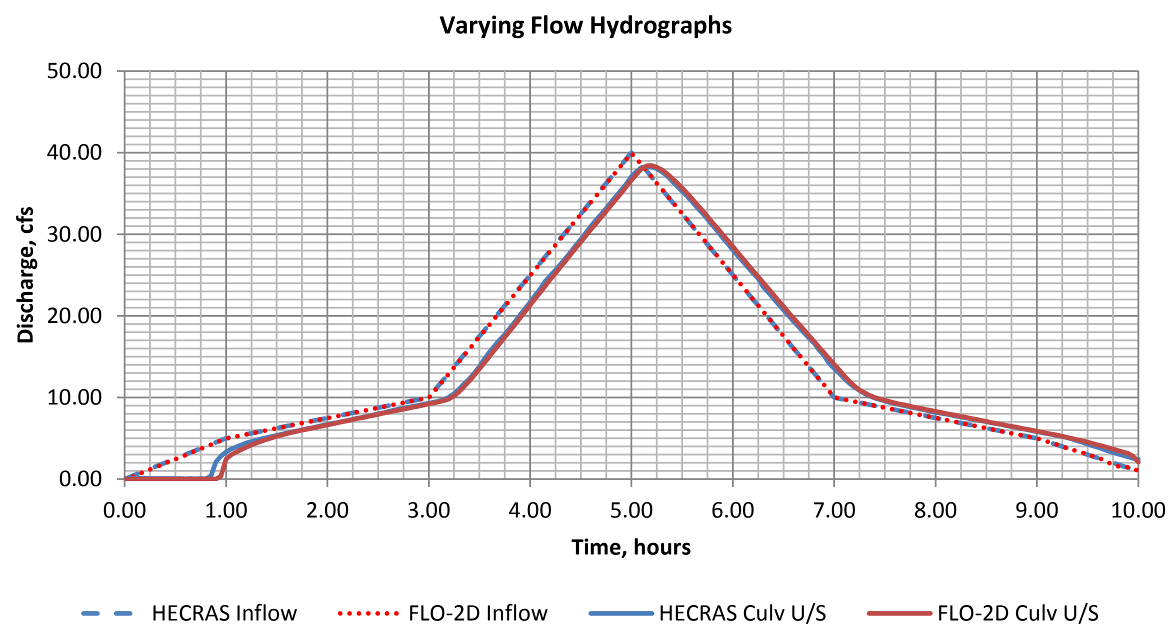


Figure 4.118 Scenario 14 Uniform Flow Hydrograph Results

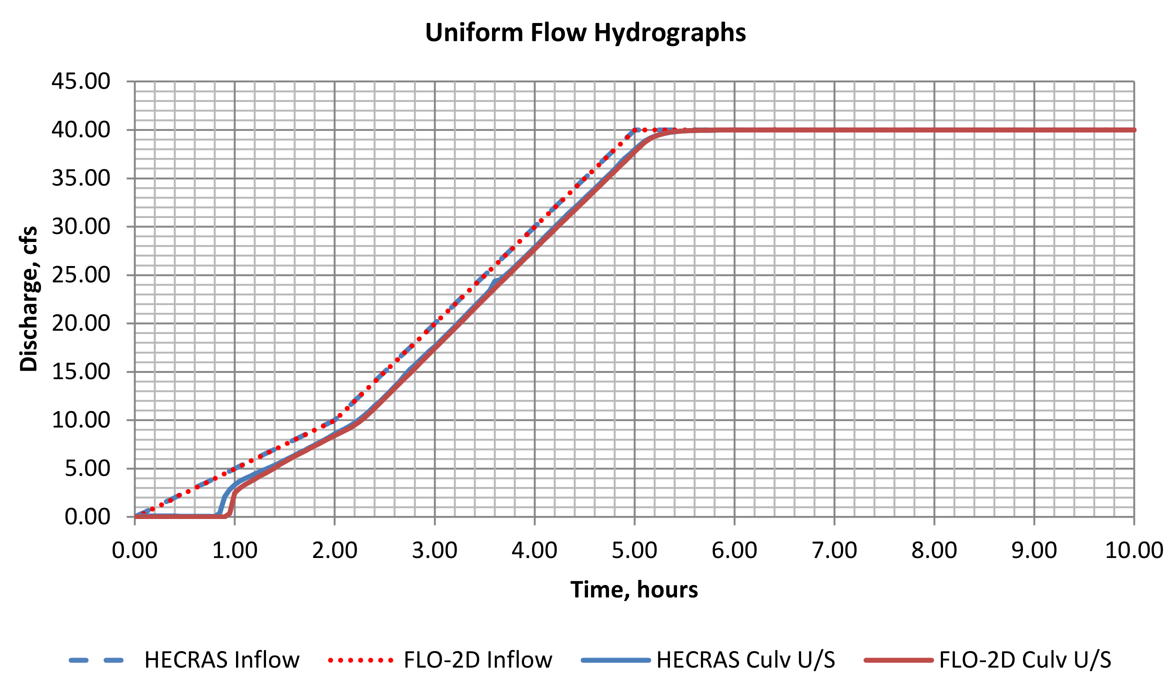


Figure 4.119 Scenario 14 Varying Flow *WSEL* Profile Results

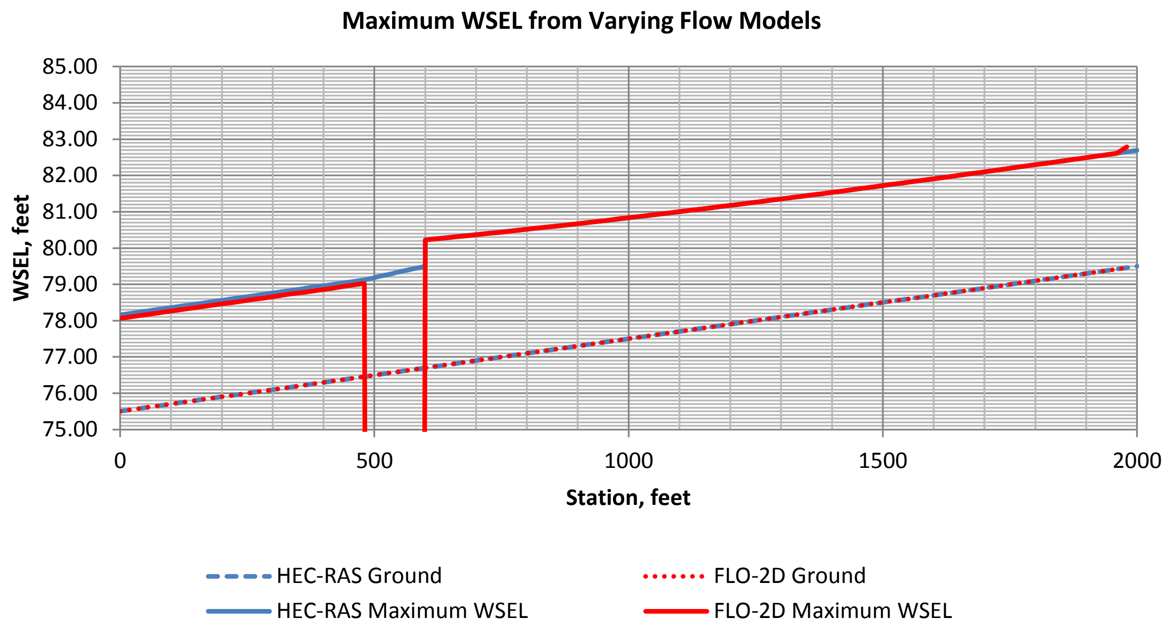


Figure 4.120 Scenario 14 Uniform Flow *WSEL* Profile Results

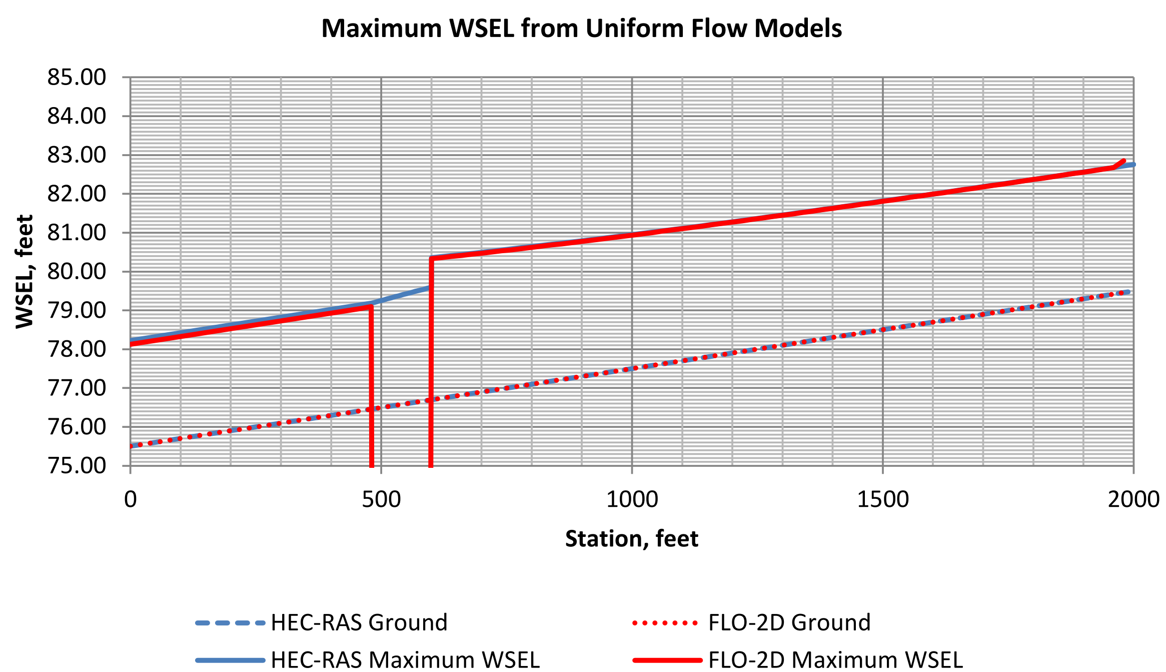
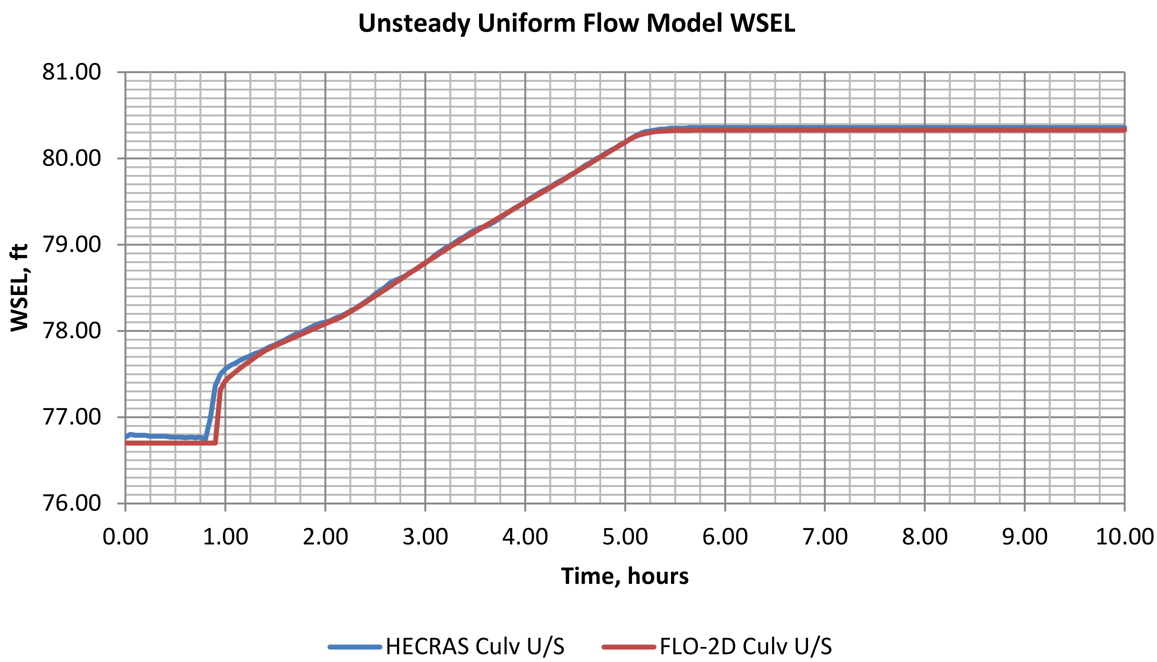


Figure 4.121 Scenario 14 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.16 Scenario 15: General Equations Channel to Floodplain (OC)

Scenario

Test of the *FLO-2D* rating table approach for a culvert connecting an upstream *1D* channel to downstream floodplain grids, operating under outlet control.

Results and Discussion

Scenario 15 is similar to Scenario 14 except that general culvert equations were used instead of the rating table approach. The culvert was assigned to connect a *1D* channel grid to a floodplain grid. The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.35. The *FLO-2D* results for the culvert at peak flow are almost identical with unsteady *HEC-RAS* for the varying flow model, except for the downstream *WSEL* being 0.1 feet lower than unsteady *HEC-RAS*, as expected because of the difference in wetted perimeter. The *FLO-2D* results for the uniform flow model are about 0.15 feet higher than unsteady *HEC-RAS* at the upstream end, and about 0.1 feet lower downstream.

The results from Benchmark 1 are shown on Figure 4.122. The general culvert equations-based results are almost identical to the rating table generated using steady state *HEC-RAS* and the unsteady *HEC-RAS* results.

The results from Benchmark 2 are shown on Figure 4.123 and Figure 4.124. The *FLO-2D* results are also nearly identical with unsteady *HEC-RAS*. There is a small numerical instability at peak for the uniform flow model. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.123 and Figure 4.124. The *RMSD* for the varying flow culvert inlet hydrograph was 0.38 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.125 and Figure 4.126. The *WSEL* profiles downstream of the culvert for the varying and uniform flow models are approximately 0.1 feet lower then *HEC-RAS*, as expected due to the difference in wetted perimeter. Upstream, the varying flow model results are nearly identical to unsteady *HEC-RAS*. The *FLO-2D* uniform flow model results upstream of the culvert are about 0.15 feet higher than *HEC-RAS* at the inlet, tapering to match farther upstream. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.125 and Figure 4.126. The *RMSD* for the varying maximum *WSEL* profile was 0.05 feet, which is a very minor deviation.

The results from Benchmark 4 are shown on Figure 4.127. The results are about 0.15 feet higher than *HEC-RAS*, as discussed above.

The minor differences between the *FLO-2D* and unsteady *HEC-RAS* results for Scenario 15 are acceptable for *FCDMC* purposes.

Table 4.35 Scenario 15 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	80.23	79.02	38.43	0.21
HEC-RAS Varying	80.23	79.12	38.35	
FLO-2D Uniform	80.51	79.09	40.01	0.03
HEC-RAS Uniform	80.36	79.18	40.00	

Figure 4.122 Scenario 15 Rating Table Floodplain to Floodplain (OC) Test Results

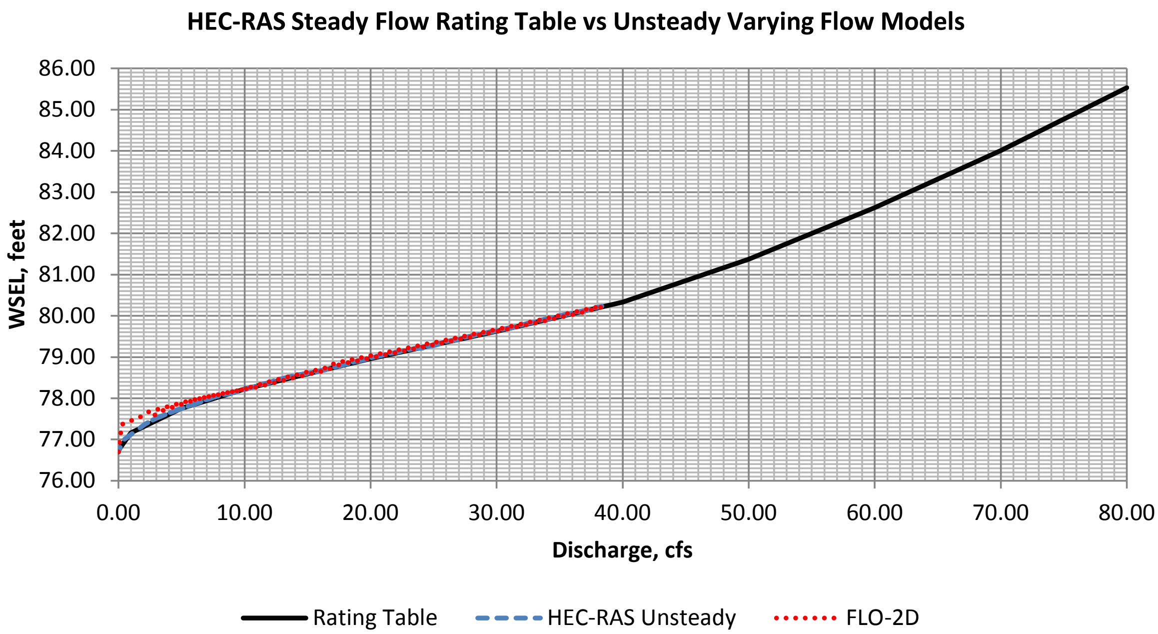


Figure 4.123 Scenario 15 Varying Flow Hydrograph Results

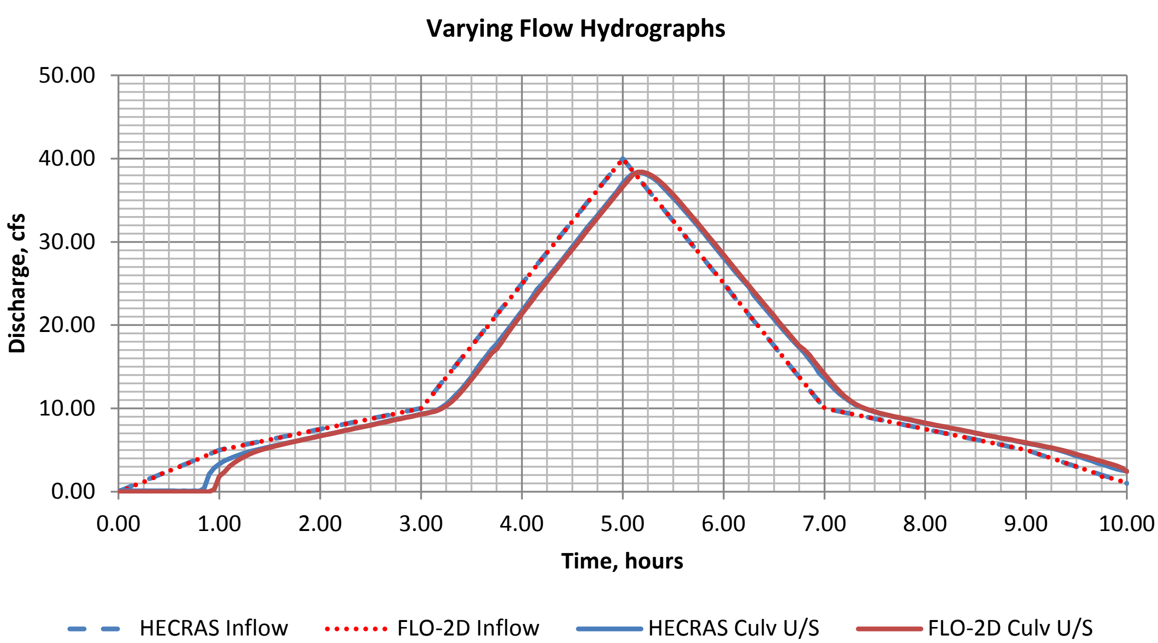


Figure 4.124 Scenario 15 Uniform Flow Hydrograph Results

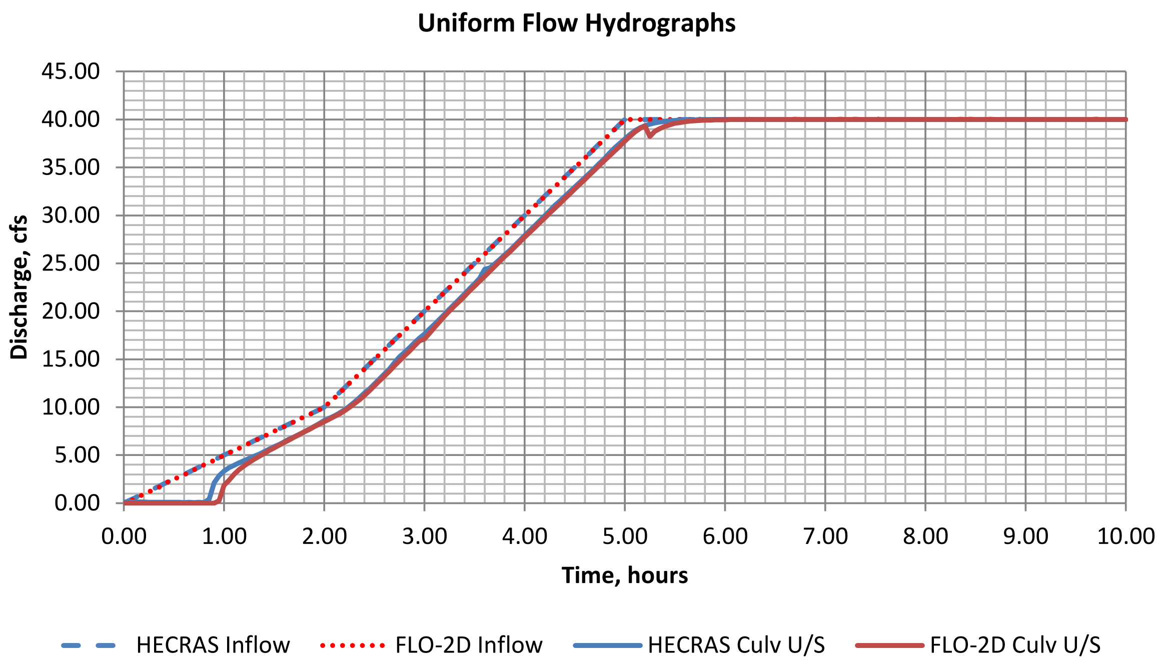


Figure 4.125 Scenario 15 Varying Flow *WSEL* Profile Results

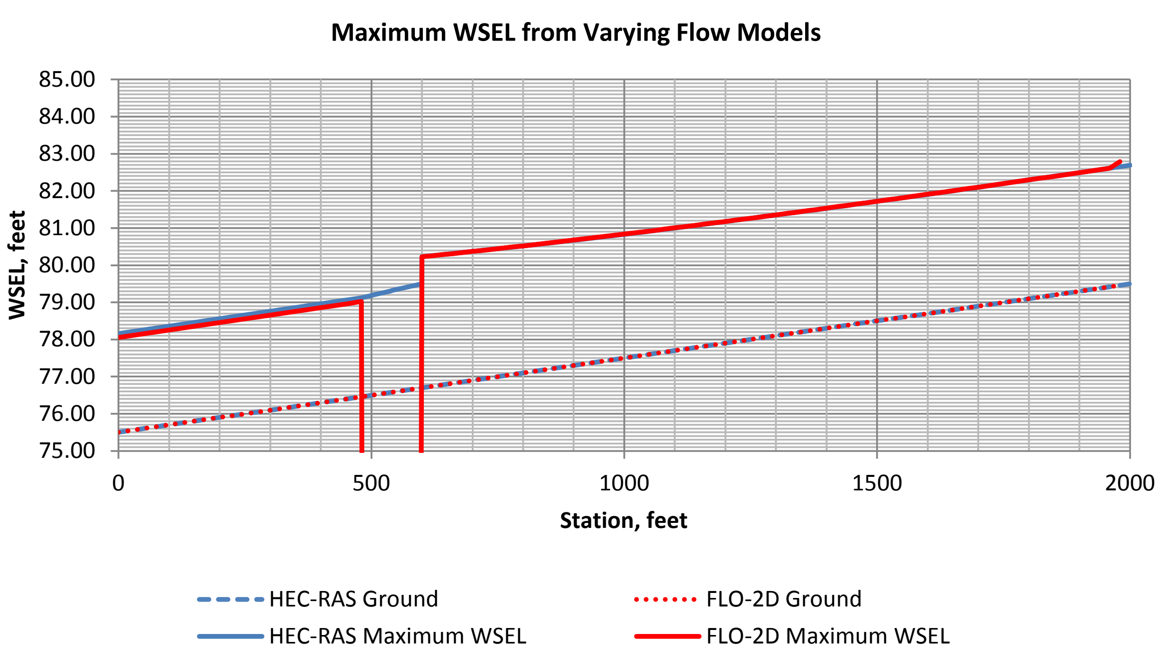


Figure 4.126 Scenario 15 Uniform Flow *WSEL* Profile Results

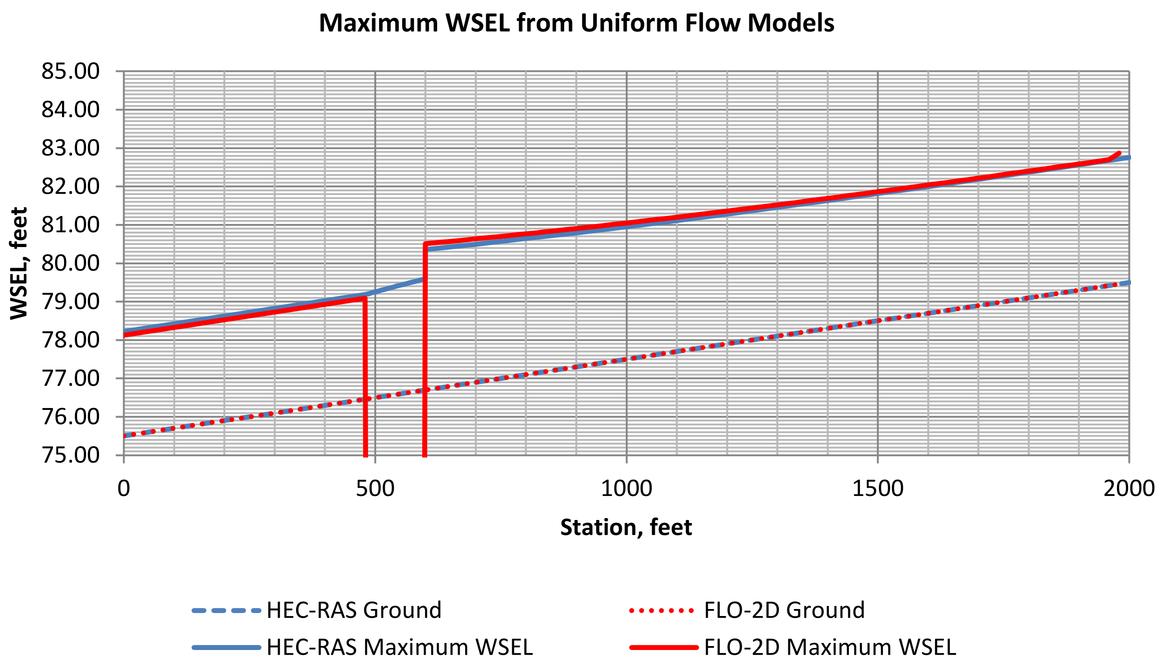
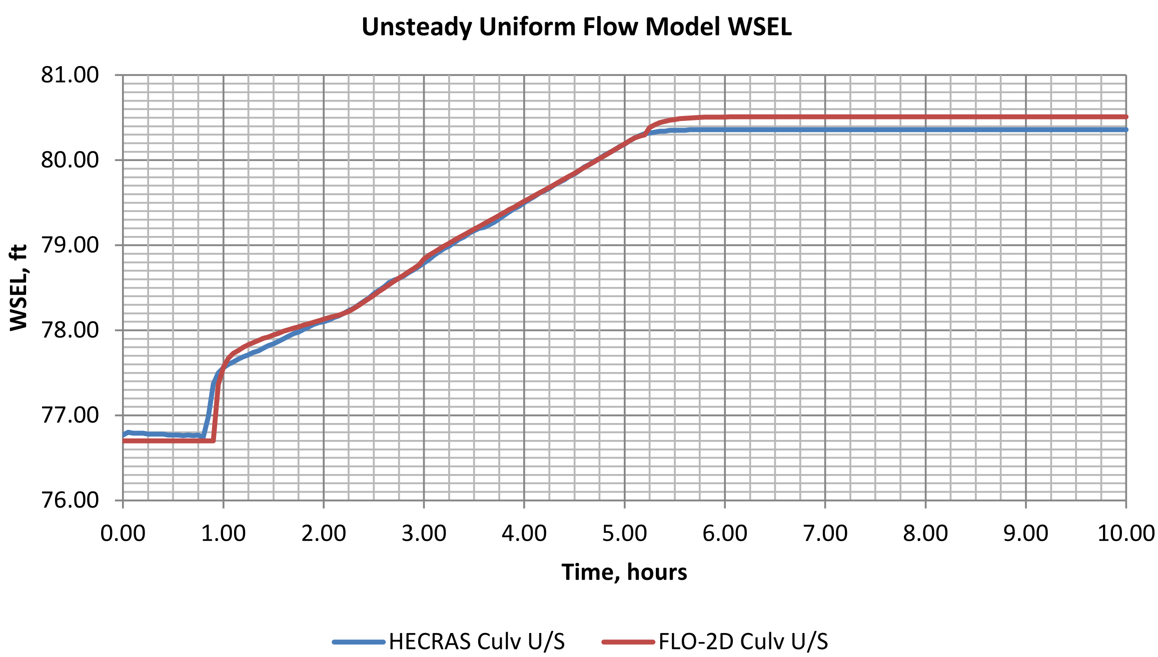


Figure 4.127 Scenario 15 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.17 Scenario 16: Rating Table Long Culvert Channel to Floodplain (IC)

Scenario

Test of the *FLO-2D* rating table approach for a long culvert connecting an upstream *1D* channel to downstream floodplain grids, operating under inlet control.

Results and Discussion

Scenario 16 is similar to Scenario 14 except that a long culvert (1,380 feet) was used and the slope increased to 0.0400 ft/ft to force inlet control, and the peak discharge was increased from 40 cfs to 80 cfs in order to submerge the inlet. A long culvert should normally function under inlet control so no additional scenarios were developed, other than this same model based on the general culvert equations (Scenario 17). The *FLO-2D* long culvert routine, triggered by defining a length and diameter greater than zero in the HYSTRUC.DAT input data file, forces the model to account for travel time in the culvert, thus translating the routed hydrograph in time to the outlet. The culvert was assigned to connect a *1D* channel grid to a floodplain grid. The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.36. The *FLO-2D* results for the culvert inlet at peak flow are almost identical with unsteady *HEC-RAS*. The *FLO-2D* results for the culvert outlet at peak flow are 0.19 feet lower than unsteady *HEC-RAS*, as expected due to the difference in wetted perimeter.

A comparison of times to peak at the culvert inlet and outlet for the varying flow hydrograph model are shown in Table 4.37. Note that *HEC-RAS* time to peak did not change from upstream to downstream through the long culvert. *HEC-RAS* reports the velocity in the culvert to be about 19 fps. For a length of 1,400 feet, the travel time is about 0.02 hours, which is what *FLO-2D* reports at peak flow. For this example, *FLO-2D* correctly translates the hydrograph to account for travel time through the pipe.

The results from Benchmark 1 are shown on Figure 4.128. The rating table-based results are almost identical to the rating table generated using steady state *HEC-RAS* and the unsteady *HEC-RAS* results.

The results from Benchmark 2 are shown on Figure 4.129 and Figure 4.130. The *FLO-2D* results are also nearly identical with unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.129 and Figure 4.130. The *RMSD* for the varying flow culvert inlet hydrograph was 0.53 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.131 and Figure 4.132. The *WSEL* profiles downstream of the culvert for the varying and uniform flow models are approximately 0.19 feet lower then *HEC-RAS*, as expected due to the difference in wetted perimeter. Upstream, the results are nearly identical to unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.131 and Figure 4.132 The *RMSD* for the varying maximum *WSEL* profile was 0.16 feet, which is a minor deviation.

The results from Benchmark 4 are shown on Figure 4.133. The *FLO-2D* results almost exactly match unsteady *HEC-RAS*.

The very minor differences between the *FLO-2D* and unsteady *HEC-RAS* results for Scenario 16 are acceptable for *FCDMC* purposes.

Table 4.36 Scenario 16 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	150.43	89.00	79.09	0.16
HEC-RAS Varying	150.42	89.19	79.22	
FLO-2D Uniform	150.52	89.01	80.00	0.00
HEC-RAS Uniform	150.52	89.20	80.00	

Table 4.37 Comparison of Culvert Travel Times at Peak Discharge

Location	Time to Peak, hours	
	FLO-2D	***HEC-RAS***
Inlet	5.05	5.05
Outlet	5.07	5.05

Figure 4.128 Scenario 16 Rating Table Floodplain to Floodplain (OC) Test Results

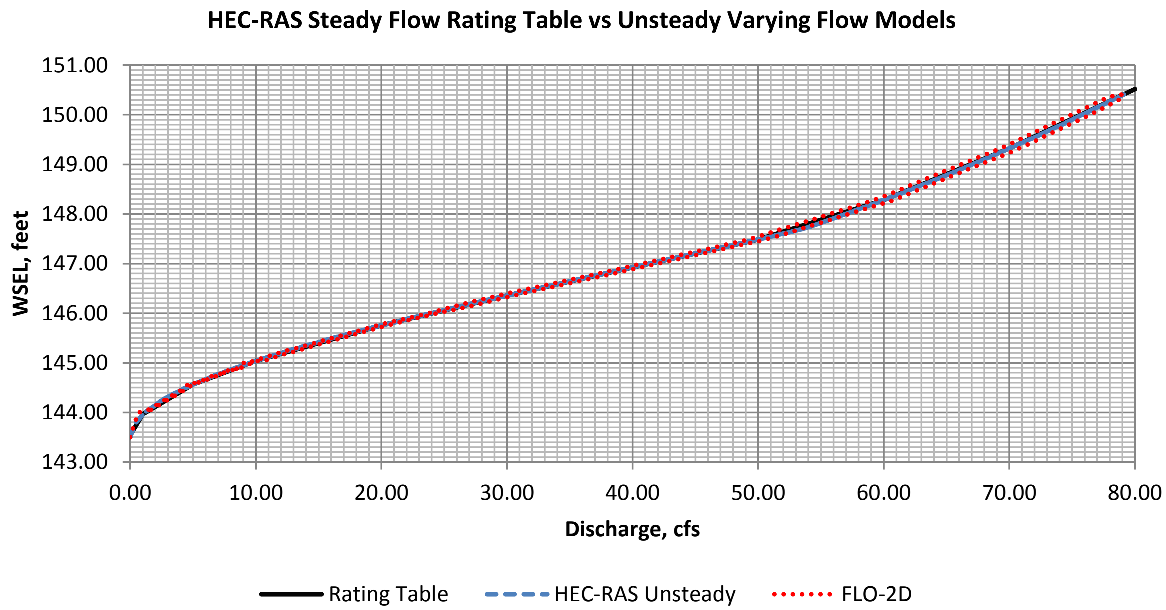


Figure 4.129 Scenario 16 Varying Flow Hydrograph Results

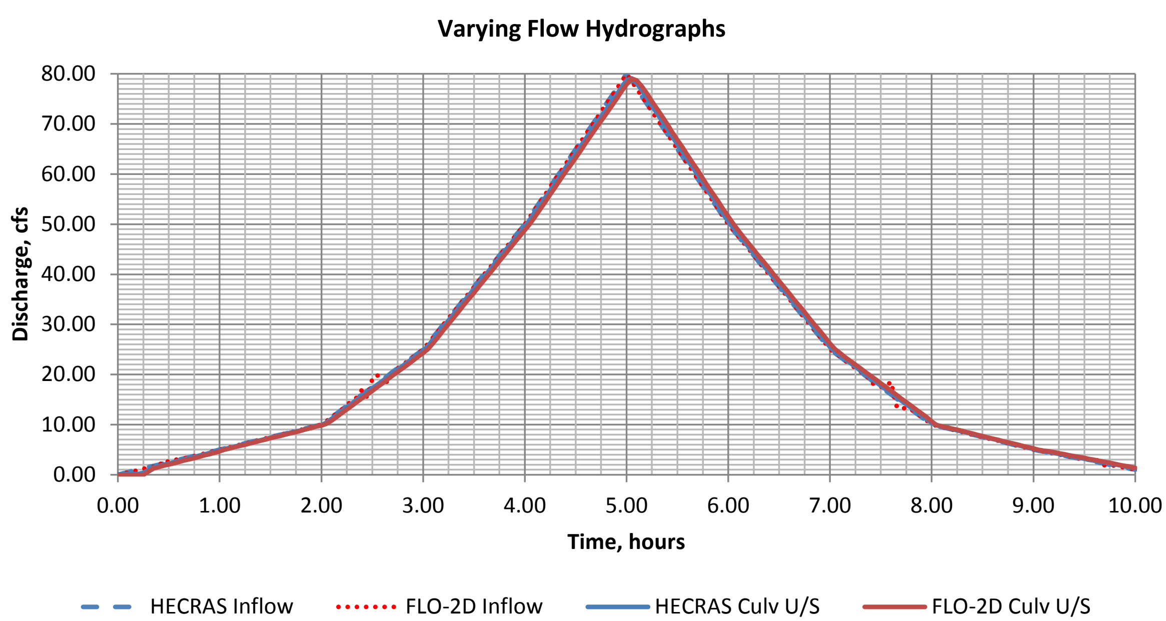


Figure 4.130 Scenario 16 Uniform Flow Hydrograph Results

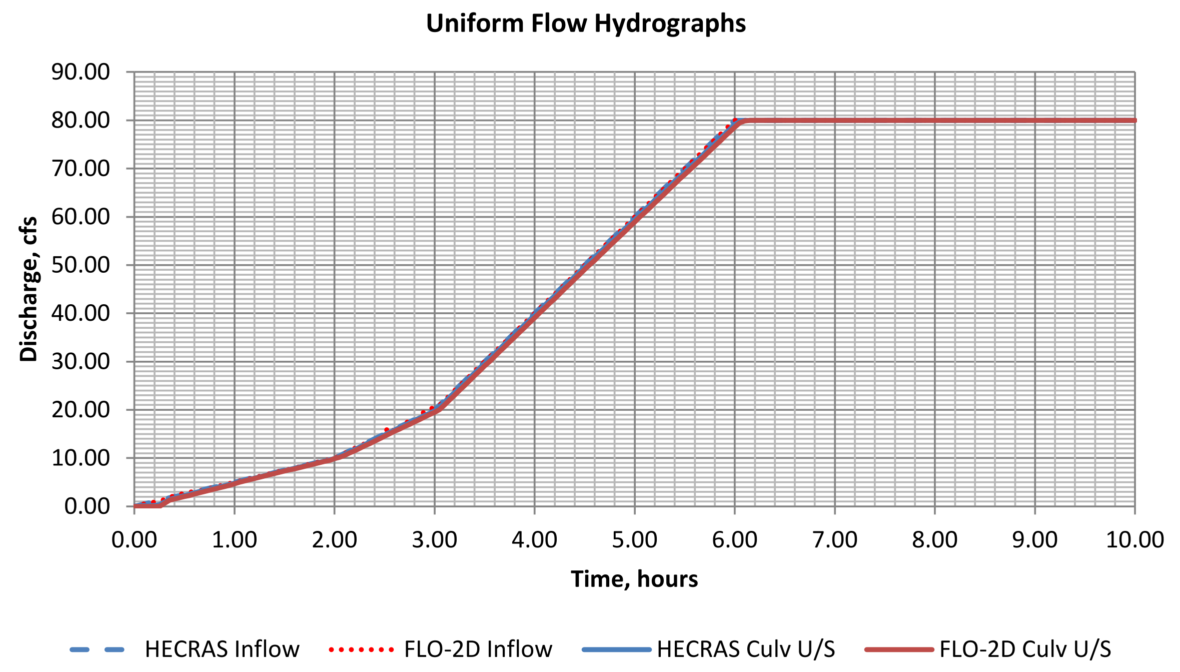


Figure 4.131 Scenario 16 Varying Flow *WSEL* Profile Results

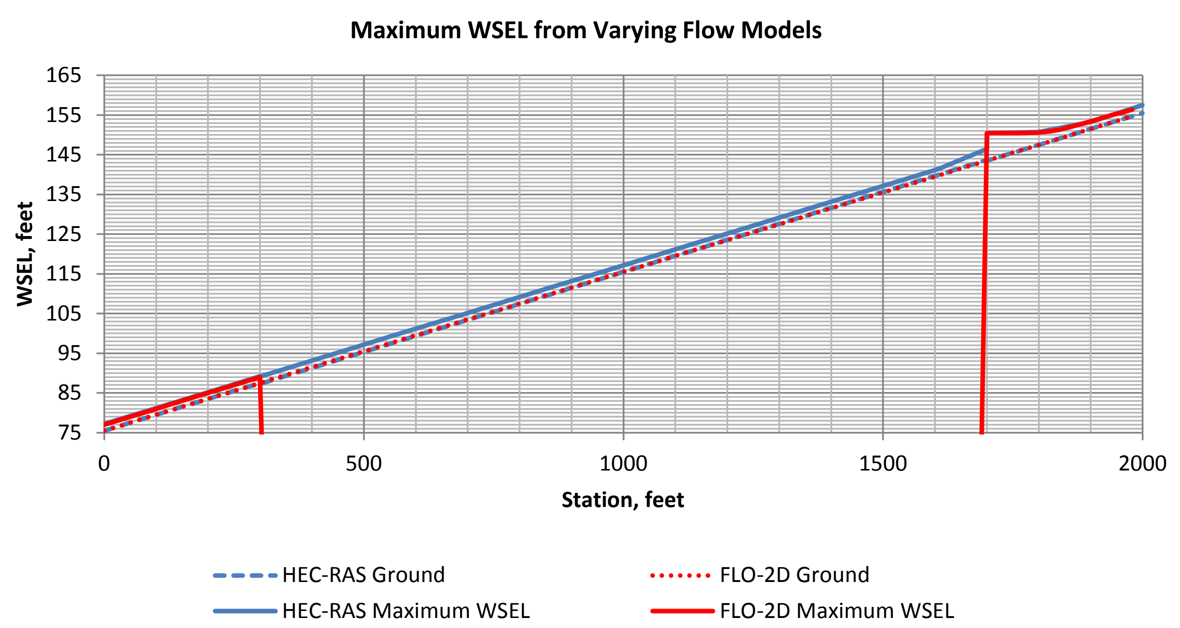


Figure 4.132 Scenario 16 Uniform Flow *WSEL* Profile Results

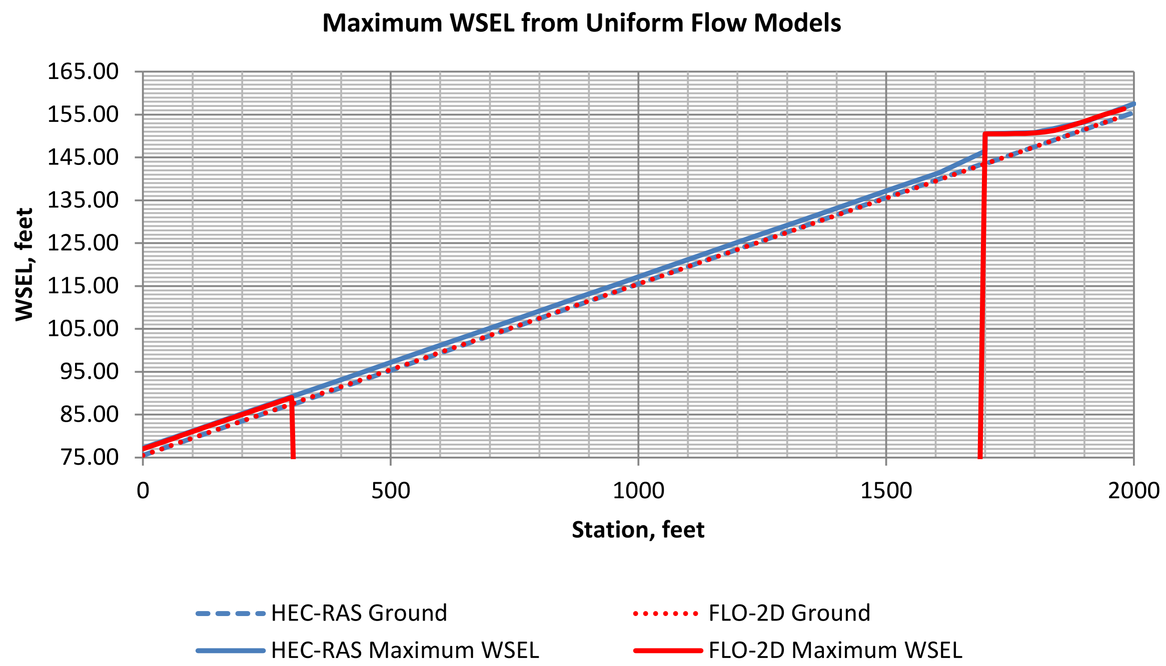
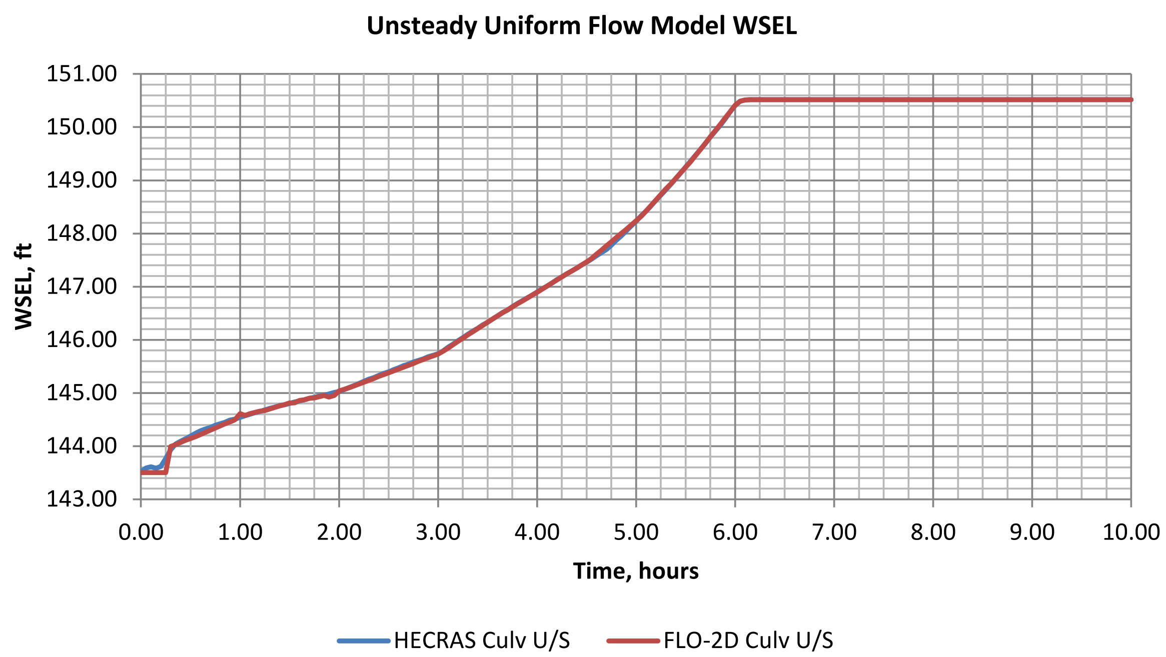


Figure 4.133 Scenario 16 Uniform Flow *WSEL* at Culvert Inlet Results



4.2.5.18 Scenario 17: General Equations Long Culvert Channel to Floodplain (IC)

Scenario

Test of the *FLO-2D* general culvert equations approach for a long culvert connecting an upstream *1D* channel to downstream floodplain grids, operating under inlet control.

Results and Discussion

Scenario 17 is similar to Scenario 16 except that the general culvert equations are used instead of the rating table method. The *FLO-2D* long culvert routine, triggered by defining a length and diameter greater than zero in the HYSTRUC.DAT input data file, forces the model to account for travel time in the culvert, thus translating the routed hydrograph in time to the outlet. The culvert was assigned to connect a *1D* channel grid to a floodplain grid. The *1D* channel was defined using the prismatic rectangular channel option as described in Section 4.2.5.1. A comparison of *WSEL* and discharge results at the culvert is shown in Table 4.38. The *FLO-2D* results for the culvert inlet at peak flow are almost identical with unsteady *HEC-RAS* for the varying flow model, but are 0.09 feet higher for the uniform flow hydrograph model. The *FLO-2D* results for the culvert outlet at peak flow are 0.19 feet lower than unsteady *HEC-RAS*, as expected due to the difference in wetted perimeter.

A comparison of times to peak at the culvert inlet and outlet for the varying flow hydrograph model are shown in Table 4.39. Note that *HEC-RAS* time to peak did not change from upstream to downstream through the long culvert. *HEC-RAS* reports the velocity in the culvert to be about 19 fps. For a length of 1,400 feet, the travel time is about 0.02 hours. *FLO-2D* reports a travel time of 0.01 hours at peak flow, which is a little low but still reasonable. For this example, *FLO-2D* does translate the hydrograph to account for travel time through the pipe, as expected.

The results from Benchmark 1 are shown on Figure 4.134. The rating table-based results are comparable to the rating table generated using steady state *HEC-RAS* and the unsteady *HEC-RAS* results.

The results from Benchmark 2 are shown on Figure 4.135 and Figure 4.136. The *FLO-2D* results are nearly identical to unsteady *HEC-RAS*. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* hydrographs at the culvert inlet as a way to quantify the significance of the minor differences noted on Figure 4.135 and Figure 4.136. The *RMSD* for the varying flow culvert inlet hydrograph was 0.54 cfs, which is a very minor deviation.

The results from Benchmark 3 are shown on Figure 4.137 and Figure 4.138. The *WSEL* profiles downstream of the culvert for the varying and uniform flow models are approximately 0.19 feet lower then *HEC-RAS*, as expected due to the difference in wetted perimeter. Upstream, the results are nearly identical to unsteady *HEC-RAS* for the varying flow hydrograph model, but about 0.09 feet higher for the uniform flow hydrograph model, as discussed above. A *RMSD* analysis was performed comparing the *FLO-2D* and *HEC-RAS* maximum *WSEL* profiles as a way to quantify the significance of the differences noted on Figure 4.137 and Figure 4.138. The *RMSD* for the varying maximum *WSEL* profile was 0.16 feet, which is a minor deviation.

The results from Benchmark 4 are shown on Figure 4.139. The *FLO-2D* results are about 0.09 feet higher than unsteady *HEC-RAS* at peak, as also noted above.

The very minor differences between the *FLO-2D* and unsteady *HEC-RAS* results for Scenario 16 are acceptable for *FCDMC* purposes.

Table 4.38 Scenario 17 Comparison of Hydraulic Structure Model Results

Model	Water Surface Elevation at Peak		Discharge through Culvert, cfs	Percent Difference
	Inlet	Outlet		
FLO-2D Varying	150.49	89.00	79.08	0.18
HEC-RAS Varying	150.42	89.19	79.22	
FLO-2D Uniform	150.61	89.01	80.00	0.00
HEC-RAS Uniform	150.52	89.20	80.00	

Table 4.39 Comparison of Culvert Travel Times at Peak Discharge

Location	Time to Peak, hours	
	FLO-2D	***HEC-RAS***
Inlet	5.05	5.05
Outlet	5.06	5.05

Figure 4.134 Scenario 17 Rating Table Floodplain to Floodplain (OC) Test Results

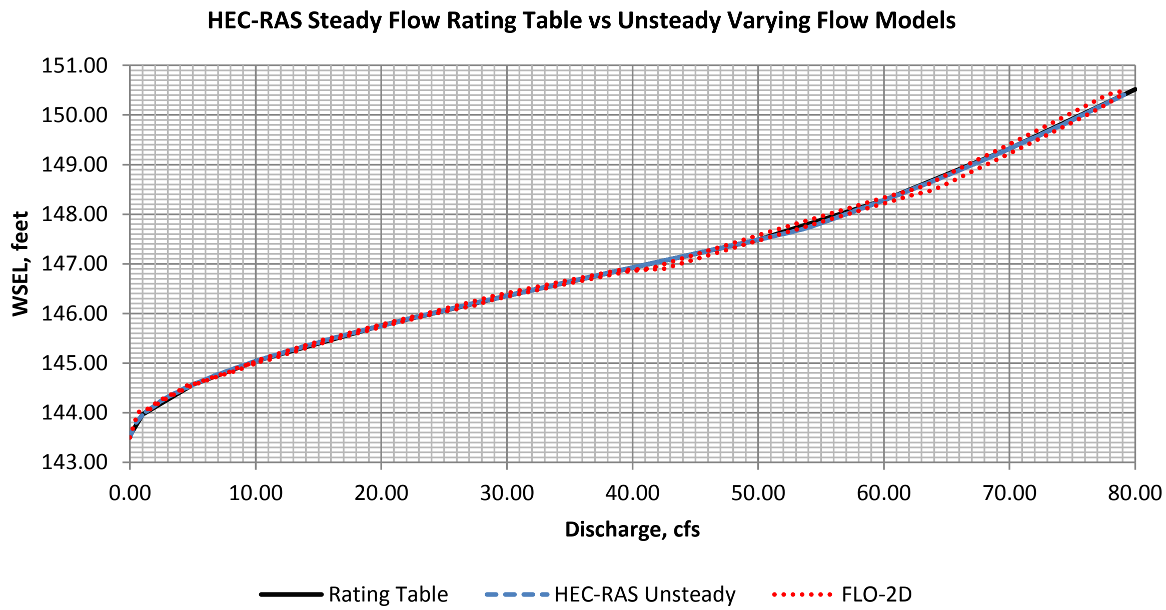


Figure 4.135 Scenario 17 Varying Flow Hydrograph Results

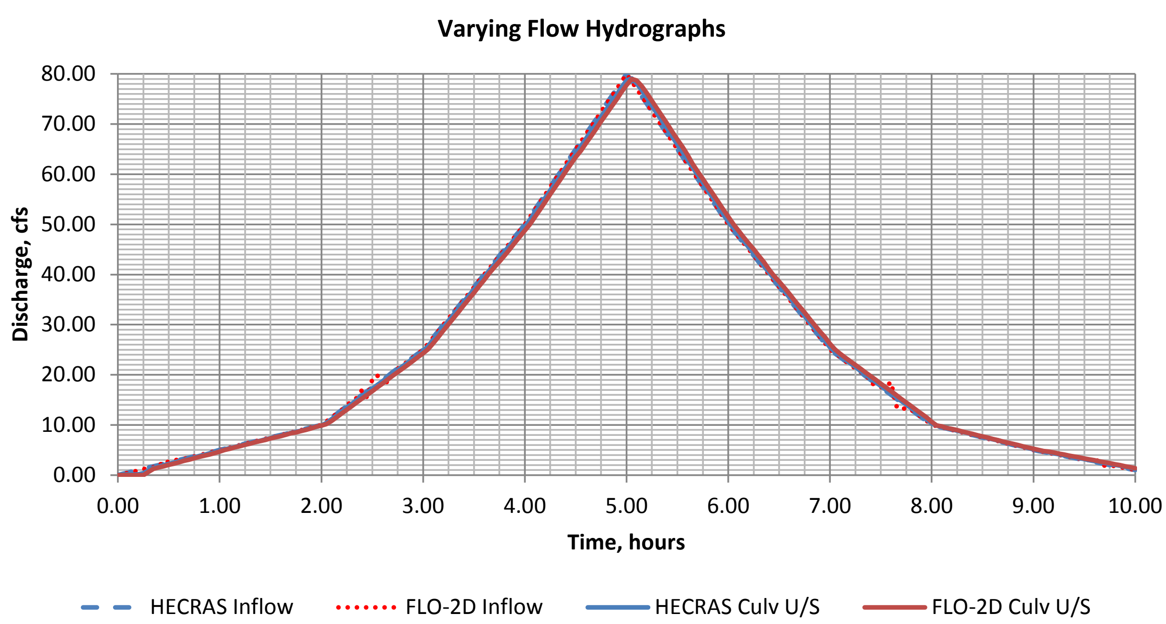


Figure 4.136 Scenario 17 Uniform Flow Hydrograph Results

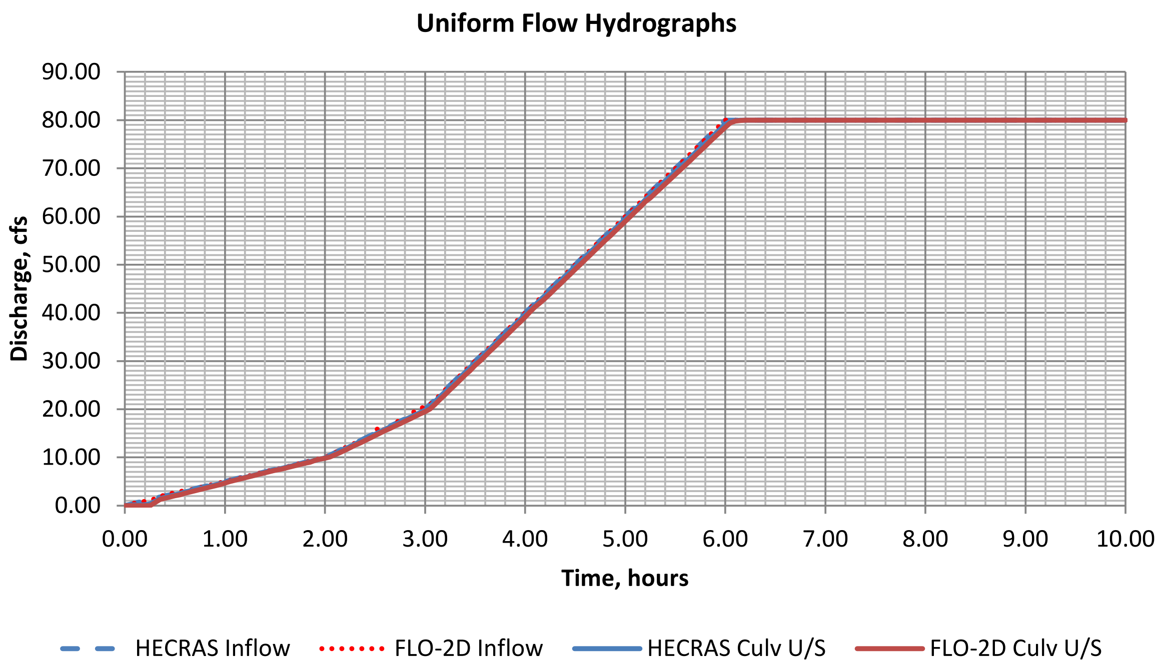


Figure 4.137 Scenario 17 Varying Flow *WSEL* Profile Results

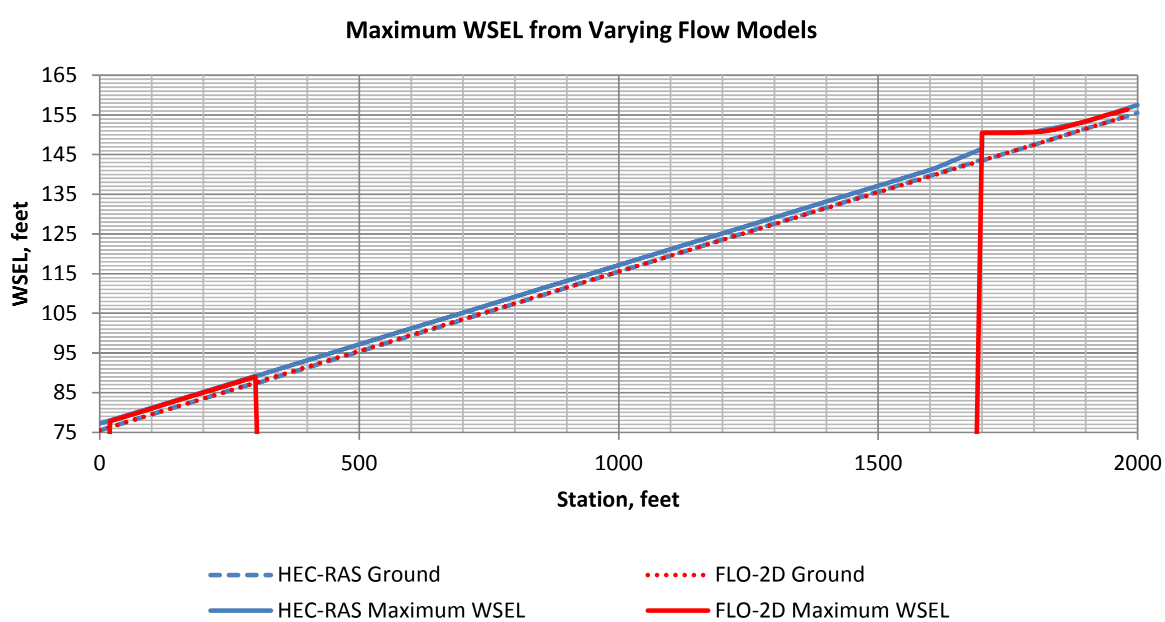


Figure 4.138 Scenario 17 Uniform Flow *WSEL* Profile Results

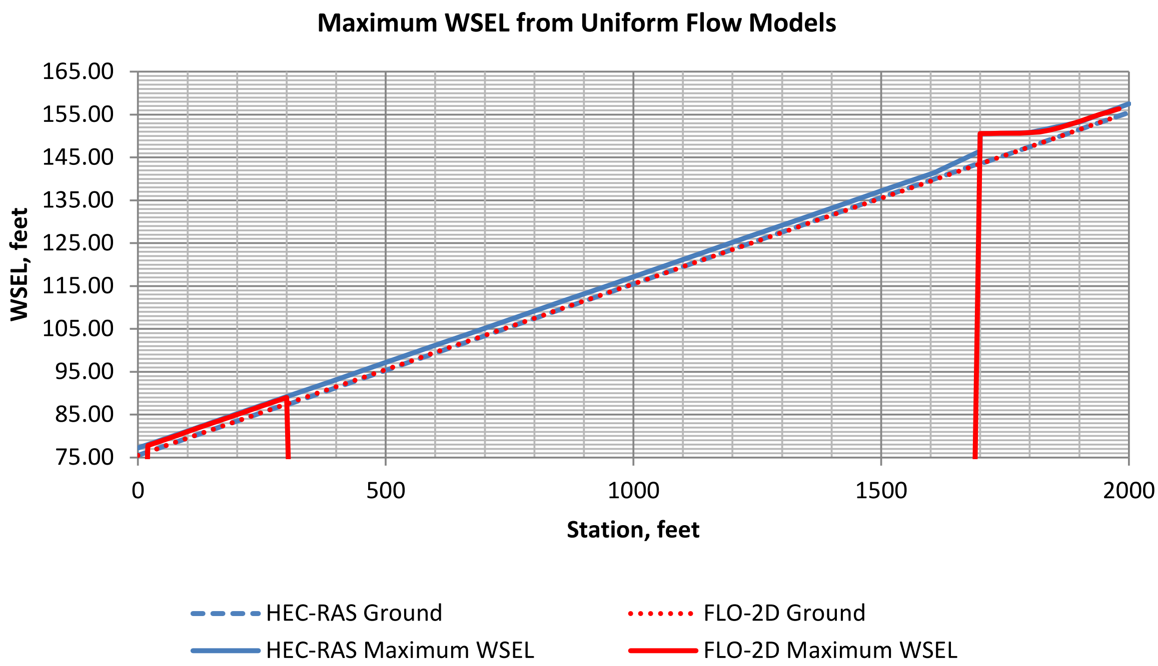
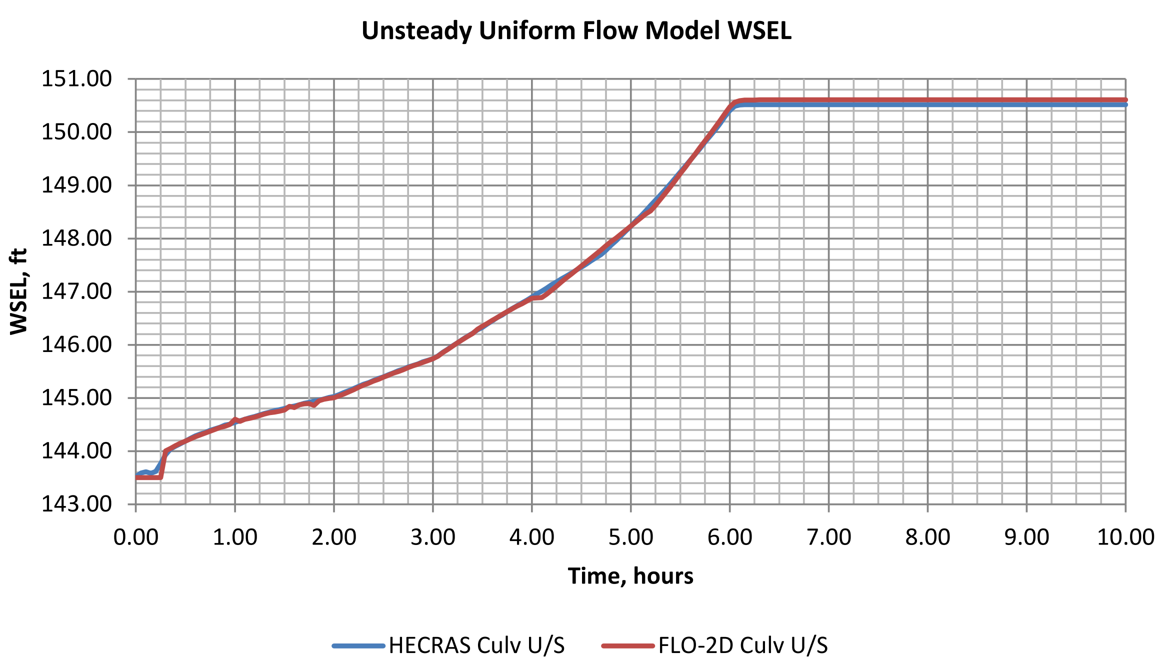


Figure 4.139 Scenario 17 Uniform Flow *WSEL* at Culvert Inlet Results



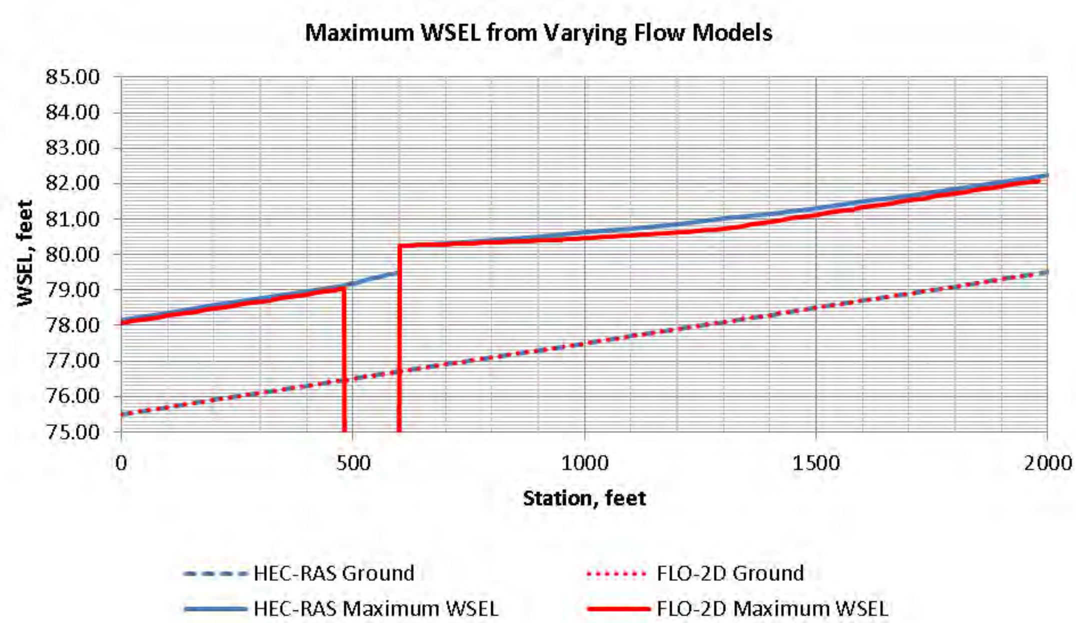
4.2.5.19 Sensitivity to Courant Value Assignment

There can be fairly significant differences in results from the *FLO-2D* hydraulic structure component depending on the selection of the Courant parameter. To quantify the differences, the hydraulic structure test scenarios were run with Courant settings of 0.1, 0.2, 0.3, 0.4 and 0.6 and the results compared. Since the test models described above are based on a Courant value of 0.2, comparisons are made with those results to provide a frame of reference. Scenarios 1, 5, 9, 12 and 13 were selected to provide comparisons for this report: Since maximum *WSEL* profiles are of particular interest to the *FCDMC* for floodplain delineation purposes, *WSEL* profiles from the varying flow hydrograph models are used for these comparisons.

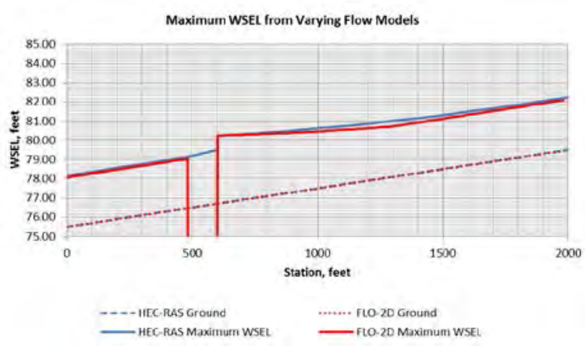
Scenario 1 comparisons are shown on Figure 4.140, Scenario 5 on Figure 4.141, Scenario 9 on Figure 4.142, Scenario 12 on Figure 4.143, and Scenario 13 on Figure 4.144. Note that for most scenarios a Courant of 0.4 to 0.6 can be used with acceptable results. The *WSEL* might be slightly higher than actual, but this is acceptable for most *FCDMC* purposes. Care must be taken when applying the general culvert equations in some situations, particularly culverts under outlet control, when connecting floodplain grids to a *1D* channel. Instabilities can result when using higher Courant numbers and both the discharge and *WSEL* results should be checked before acceptance.

Figure 4.140 Scenario 1 *WSEL* Profile Results Comparisons for Different Courant Settings

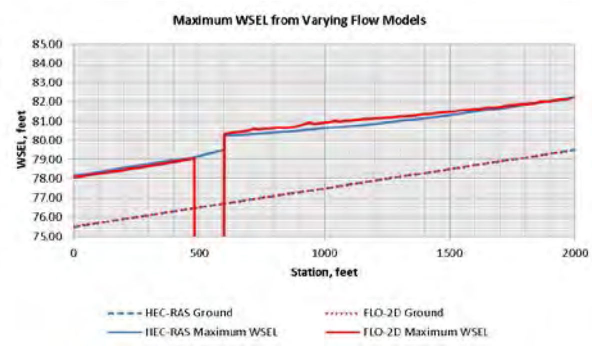
Courant = 0.2



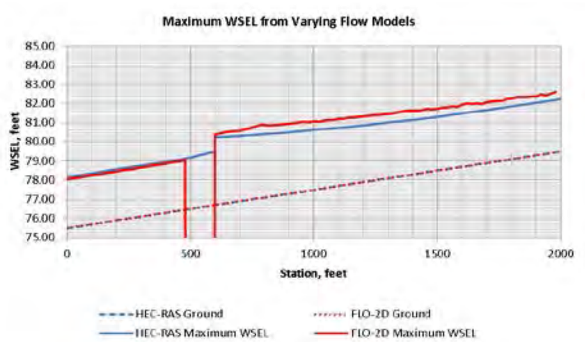
Courant = 0.1



Courant = 0.3



Courant = 0.4



Courant = 0.6

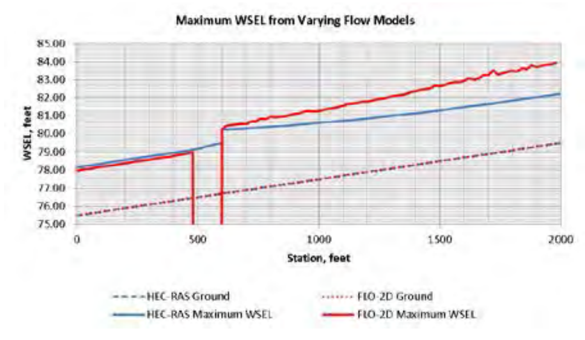
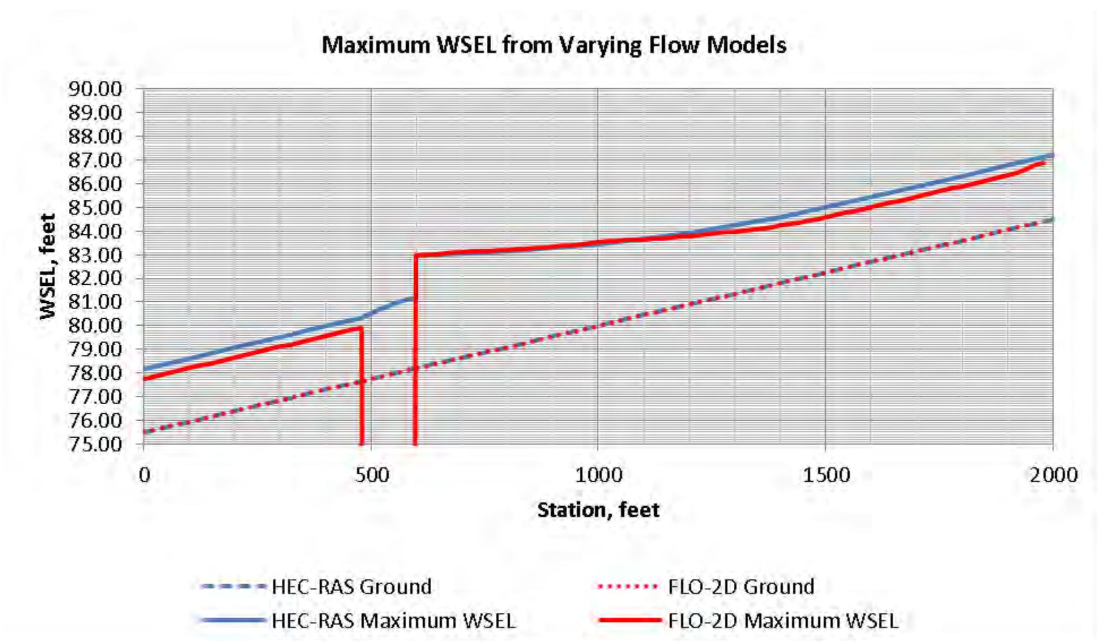


Figure 4.141 Scenario 5 *WSEL* Profile Results Comparisons for Different Courant Settings

Courant = 0.2



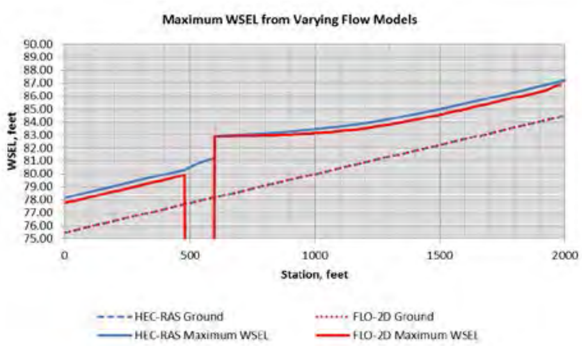
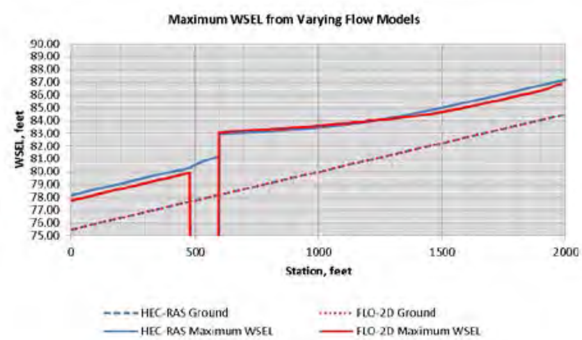
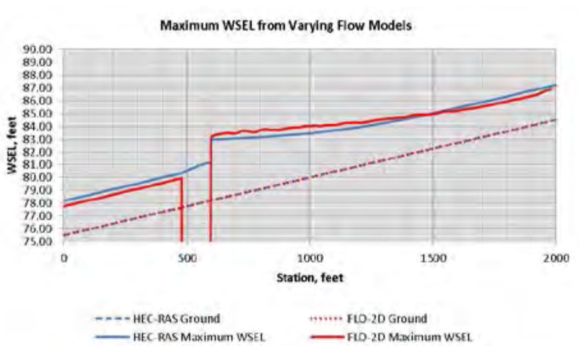
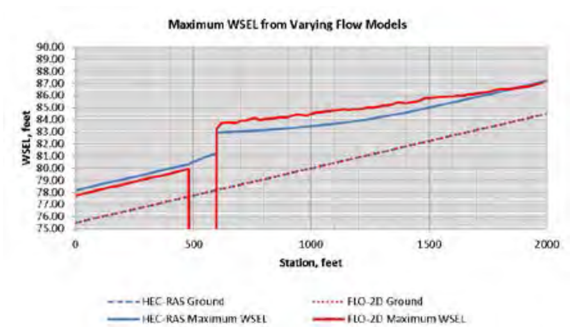
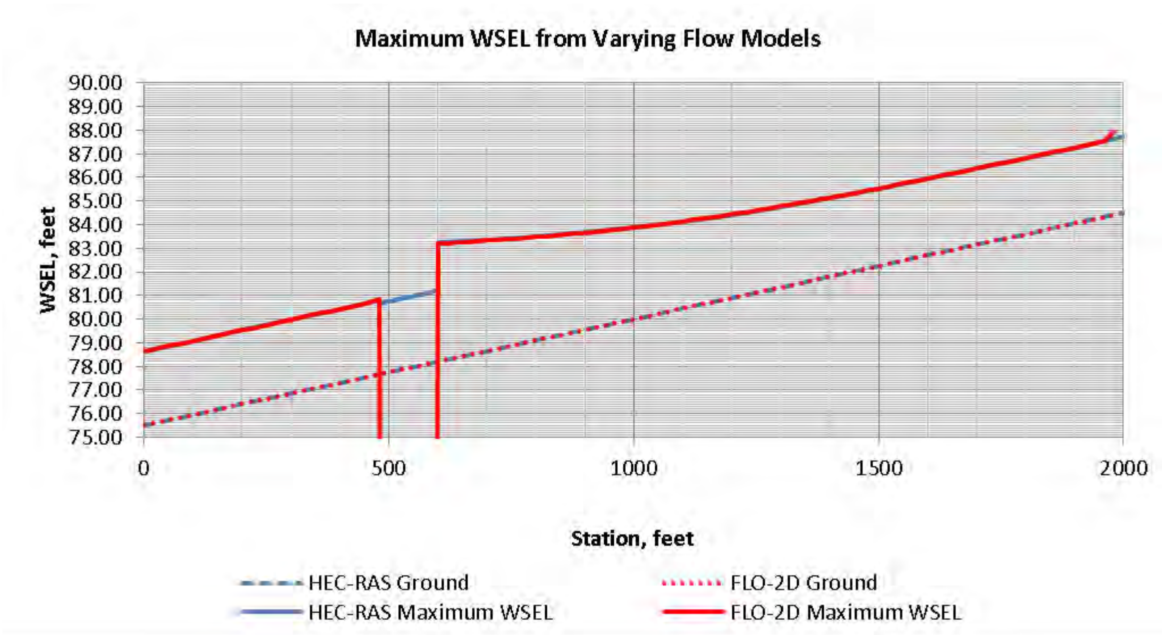
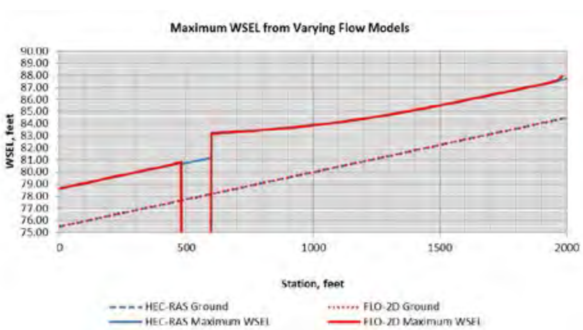
<table>
<tr><td>Courant = 0.1</td><td>Courant = 0.3</td></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td>Courant = 0.4</td><td>Courant = 0.6</td></tr>
<tr><td>
</td><td>
</td></tr>
</table>

Figure 4.142 Scenario 9 *WSEL* Profile Results Comparisons for Different Courant Settings

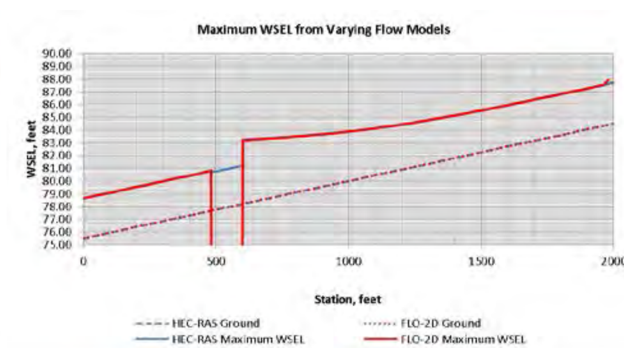
Courant = 0.2



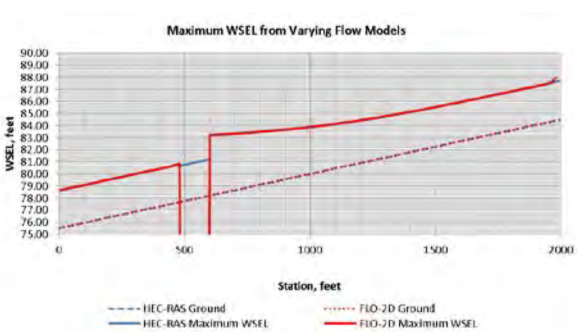
Courant = 0.1



Courant = 0.3



Courant = 0.4



Courant = 0.6

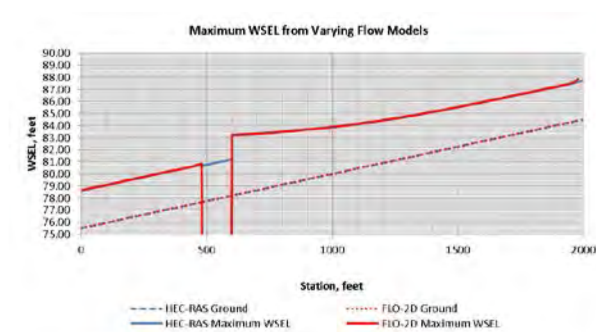
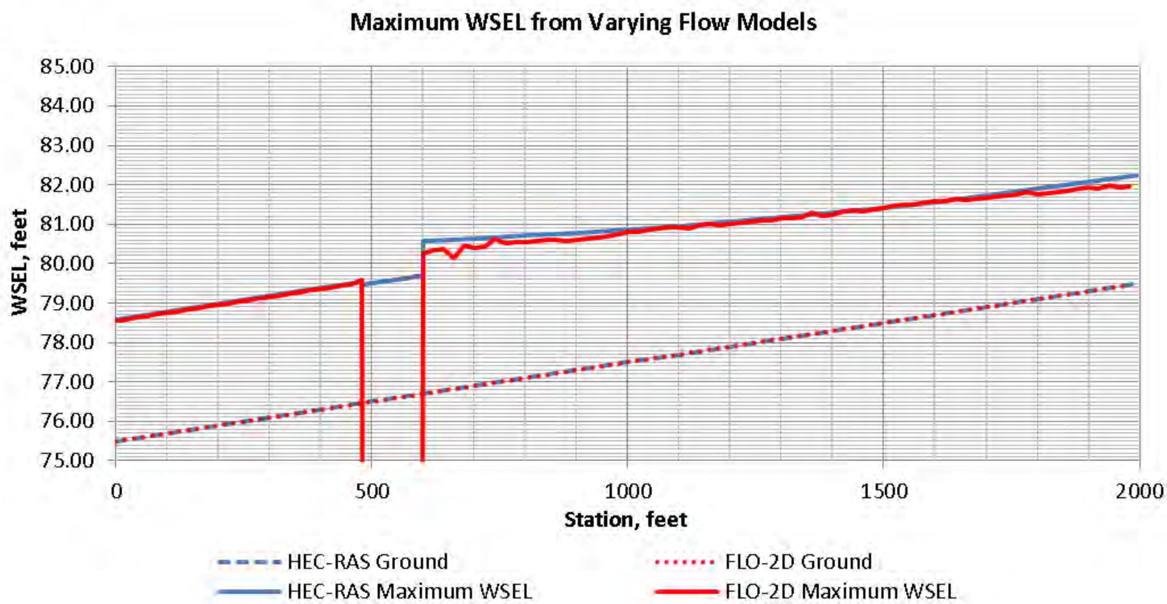
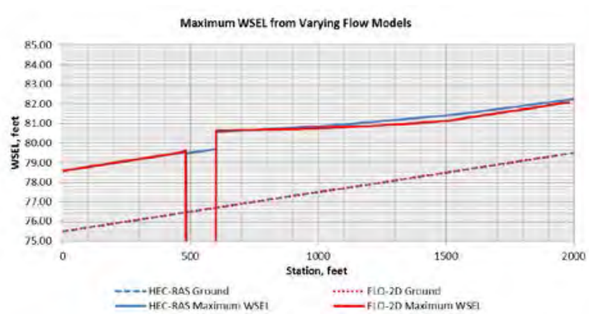


Figure 4.143 Scenario 12 *WSEL* Profile Results Comparisons for Different Courant Settings

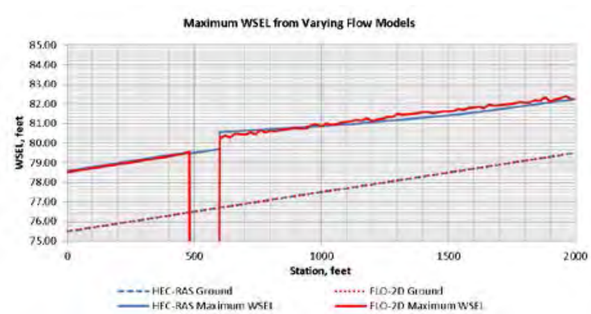
Courant = 0.2



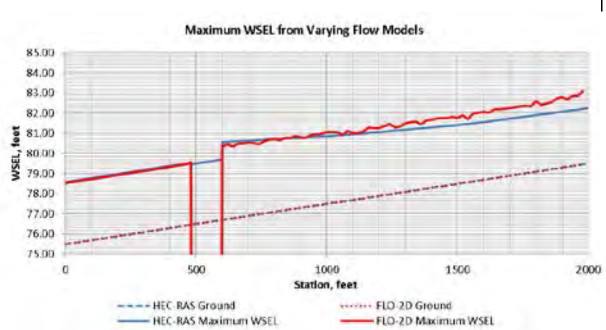
Courant = 0.1



Courant = 0.3



Courant = 0.4



Courant = 0.6

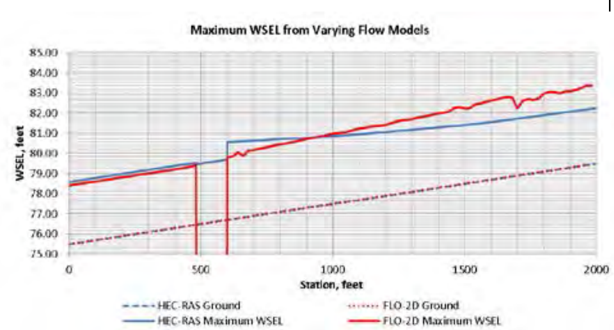
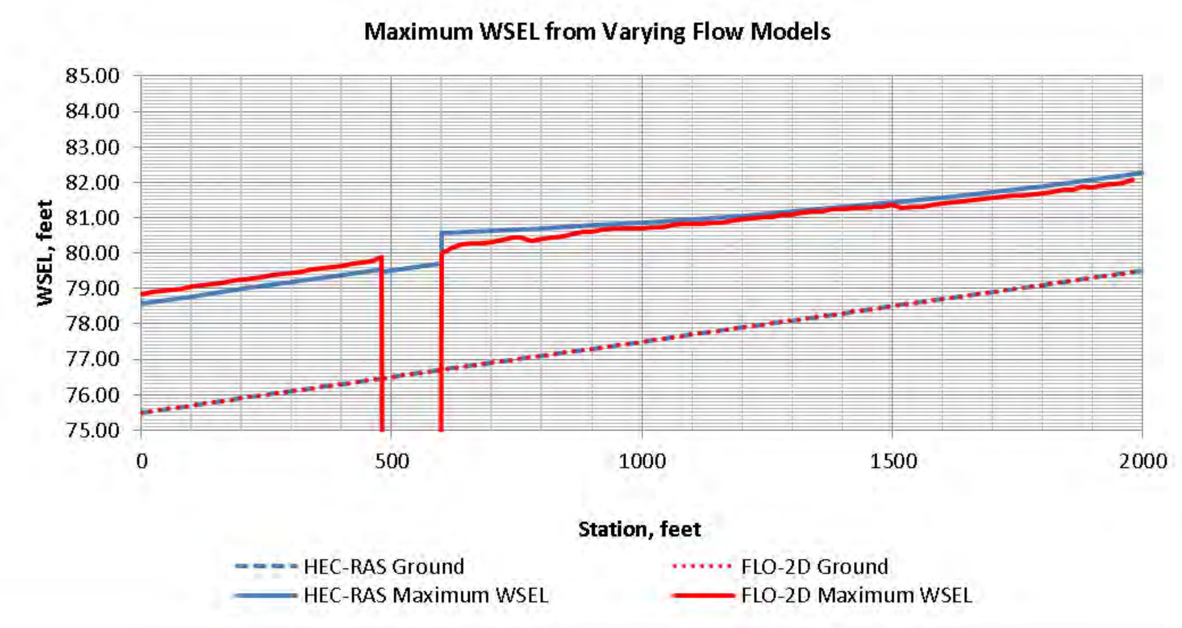
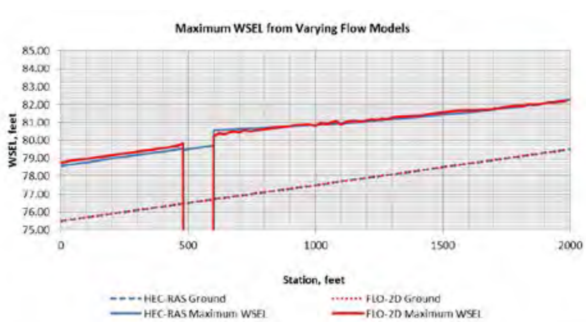


Figure 4.144 Scenario 13 *WSEL* Profile Results Comparisons for Different Courant Settings

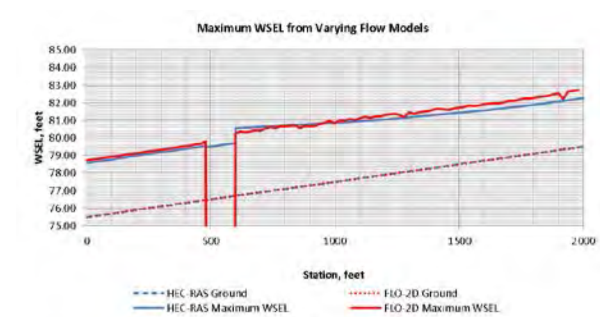
Courant = 0.2



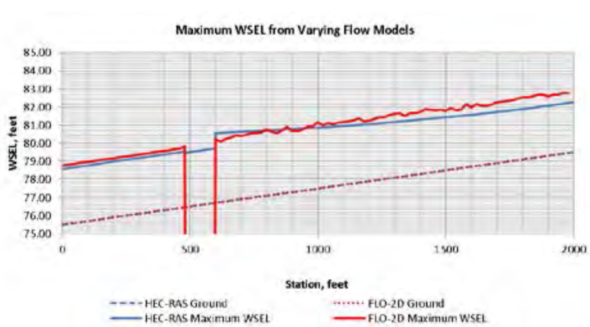
Courant = 0.1



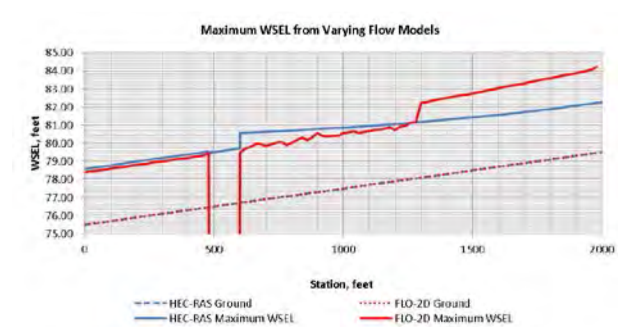
Courant = 0.3



Courant = 0.4



Courant = 0.6



4.2.5.20 Summary and Conclusions

The hydraulic structure component applied using the rating table option works very well for the scenarios tested and the results check very closely with *HEC-RAS*. The Courant parameter may need to be lowered below 0.6 in many instances for models containing hydraulic structures but it is a good initial setting. A Courant of 0.4 or smaller may be necessary for final model runs. Use of the general culvert equations with floodplain grids under outlet control may produce more numeric instability than the rating table option. In all cases, the modeler should carefully examine the results of the hydraulic structure routine for instability, reasonableness and consistency and make adjustments as necessary.

Based on these test scenarios, the *FLO-2D* hydraulic structure component appropriately models culvert hydraulics and is acceptable for *FCDMC* modeling purposes.

4.2.6 Multiple Channels

4.2.6.1 General

The purpose of this section is to verify that hydraulic calculations for the overland multiple channel flow component are performed correctly. This function is used to simulate channelized flow in rills and gullies that are present within a grid element. The results of the *FLO-2D* model are compared to a *HEC-RAS* model with similar characteristics. Refer to Section 2.4.7.3 for a description of how the multiple channel component is intended to work.

4.2.6.2 Scenario 1

This scenario is a test of the multiple channel component for the case where the channel width is not allowed to expand. The discharge is set to keep flow within the defined channel, and the hydraulics are then verified using a *HEC-RAS* model. A single inset channel defined using the multiple channel component is used to connect upstream and downstream floodplain grids that are configured to form a flume-type channel. The grid ground elevations defined in FPLAIN.DAT containing the multiple channel are raised so that the defined multiple channel is on a continuous uniform slope connecting the upstream and downstream flumes. The *FLO-2D* model uses a 10-foot grid size and has the following characteristics:

Upstream and downstream flume:

1. Base Width: 10 feet for *FLO-2D* and 4.142 feet for *HEC-RAS*
2. Flume depth 10 feet (adjacent grids are 10 feet higher than the flume channel bottom)
3. Slope: uniform at 4%
4. *n*-value: 0.040

5. Length: 110.0 feet

The multiple channel component has the following parameter settings:

1. Incremental width by which multiple channels will be expanded: 0
2. Multiple channel width: 3.0 feet
3. Maximum depth: 5.0 feet (floodplain grids containing the multiple channel are 5 feet lower than the adjacent grid.)
4. Number of multiple channel assignments: 1
5. Multiple channel *n*-value: 0.040
6. Minimum slope: 0 ft/ft
7. Maximum slope: 0 ft/ft

A *HEC-RAS* model with the above characteristics was built using cross sections at 10 foot intervals and a uniform peak discharge of 5 cfs. A plan view of the *HEC-RAS* and *FLO-2D* models are shown on Figure 4.145 and Figure 4.146, respectively. The *HEC-RAS* model has a base width of 4.142 feet to simulate the *FLO-2D* flumes upstream and downstream of the multiple channel test reach. This is necessary to account for the difference in wetted perimeter (refer to Section 4.2.2.2). In the multiple channel test reach, the base width is set to 3 feet.

Figure 4.145 Flume Model Using *HEC-RAS* for Multiple Channel Flow Component

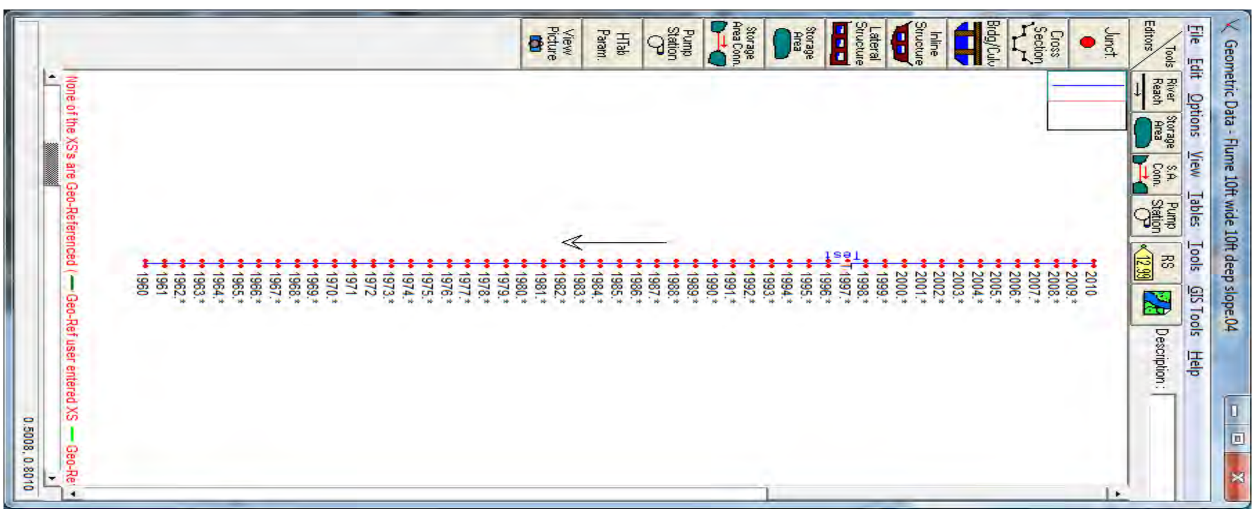
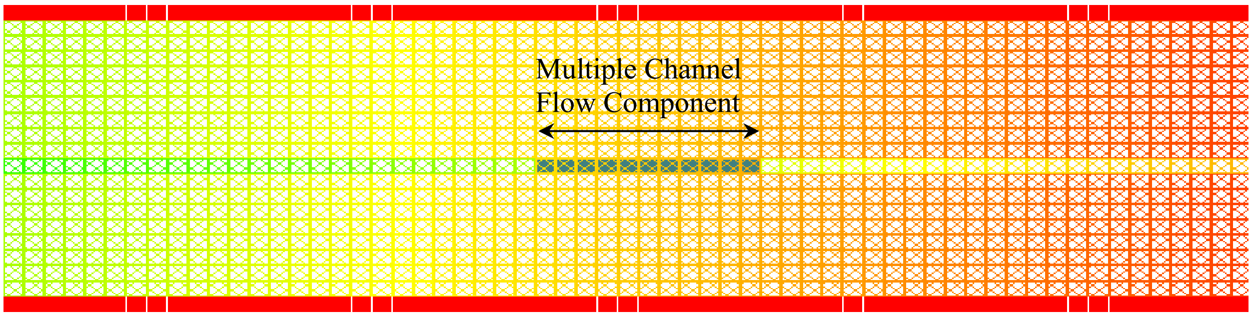


Figure 4.146 *FLO-2D* Model for Multiple Channel Flow Component



In order to be able to directly compare the *FLO-2D* results with *HEC-RAS*, the *FLO-2D* functions that adjust *n*-value are turned off. The limiting Froude number was set to zero in order to avoid any *n*-value adjustments. Also, the depth-variable *n*-value routine was turned off (AMANN in CONT.DAT). The *HEC-RAS* model was run in subcritical mode. Important *FLO-2D* input control settings used were:

1. Limiting Froude number = 0
2. Shallow *n* = 0.0
3. AMANN = -99 (turned off)
4. Floodplain COURANTFP = 0.6
5. *TOL* = 0.001

The results from the *FLO-2D* and *HEC-RAS* models are compared in Table 4.40. The results are comparable but slightly different between the two models. The slight differences are due to *FLO-2D* using a wetted perimeter that only includes the channel base width, not the sides. As an additional check, the velocity and area values were used to compute the discharges shown in the last columns.

The conclusion is that *FLO-2D* multiple channel method performs hydraulic calculations in an acceptable manner for a small shallow depth fixed width channel.

Table 4.40 Comparison of *FLO-2D* Multiple Channel Component with *HEC-RAS*									
***HEC-RAS* River Station**	***FLO-2D* Grid Number**	**Depth**		**Velocity**		**Area**		**Calculated Discharge**	
		RAS	***2D***	**RAS**	***2D***	**RAS**	***2D***	**RAS**	***2D***
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**
1969	1969	0.45	0.41	3.72	4.07	1.34	1.23	4.98	5.01
1967	1967	0.45	0.41	3.69	4.09	1.35	1.22	4.98	5.00
1963	1963	0.45	0.41	3.73	4.09	1.34	1.22	5.00	5.00

4.2.6.3 Scenario 2

This scenario is a test of the multiple channel component for the case where the channel width is not allowed to expand and the discharge is high enough that the input multiple channel maximum depth is exceeded. The model parameters are the same as Scenario 1. The hydraulics are then verified using a *HEC-RAS* model and the results listed in Table 4.41. Note that the differences between *HEC-RAS* and *FLO-2D* are much greater for this scenario than for Scenario 1. The differences are due to *FLO-2D* using a wetted perimeter that only includes the channel base width, not the sides.

The conclusion is that the *FLO-2D* multiple channel method is not an acceptable approach for modeling channels that have high flow rates and depths greater than two (2) feet, particularly if the assigned multiple channel depth will be exceeded and the channel is not allowed to expand.

Table 4.41 Comparison of *FLO-2D* Multiple Channel Component with *HEC-RAS*									
***HEC-RAS* River Station**	**FLO-*2D* Grid Number**	**Depth**		**Velocity**		**Area**		**Calculated Discharge**	
		RAS	***2D***	**RAS**	***2D***	**RAS**	***2D***	**RAS**	***2D***
(1)	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	**(9)**	**(10)**
1969	1969	4.19	2.43	7.95	13.91	12.57	7.29	100.0	101.4
1967	1967	4.18	2.46	7.97	13.57	12.55	7.37	100.0	100.0
1963	1963	4.11	2.46	8.11	13.56	12.33	7.38	100.0	100.1

4.2.6.4 Scenario 3

This scenario is a test of the multiple channel component for the case where the channel width is allowed to expand. The goal is to test than the expansion does occur as expected. However, due to the complexity of the dynamic calculations involved, verification of the hydraulic calculations was not performed. The model parameters are the same as Scenario 1 except that the expansion incremental width (variable WMC) was set to 0.1 feet. The channel does expand as expected during the modeling and is reported in the MULTCHN.OUT.

4.2.6.5 Summary and Conclusions

The multiple channel component is acceptable for *FCDMC* purposes with the following limitations:

1. Fixed channel with applications: The component should only be applied for shallow channels (<2 ft in depth) with small flow rates that will not exceed the assigned depth.
2. Channels where the width is allowed to expand: The component should only be applied where the *FLO-2D* wetted perimeter assumption does not significantly affect the model results.

4.2.7 *EPA-SWMM* Component

4.2.7.1 General

The purpose of this section is to verify that hydraulic calculations for modeling storm drain systems are done appropriately by comparing the *FLO-2D* results with hand calculations or other methods. Due to the complexity of the storm drain inlet, outlet, outfall and manhole junction options available, forty three (43) scenarios were tested in order to thoroughly evaluate the *FLO-2D-SWMM* interface. Verification models were prepared for the following scenarios:

1. Scenario 1: Inlet Type 1, Weir Flow, *HGL* below Soffit
2. Scenario 2: Inlet Type 1, Orifice Flow, *HGL* below Soffit
3. Scenario 3: Inlet Type 1, Weir/Orifice Transition, *HGL* below Soffit
4. Scenario 4: Inlet Type 1, Orifice Flow, Ponded Depth, *HG*L below Soffit
5. Scenario 5: Inlet Type 1, Orifice Flow, Pipe Surcharged, *WSEL* > *HGL* > Inlet
6. Scenario 6: Inlet Type 1, Scenario 5 with Bypass Flow
7. Scenario 7: Inlet Type 1, Weir Flow, *HGL* below Soffit, with Curb
8. Scenario 8: Inlet Type 2, Weir Flow, *HGL* below Soffit
9. Scenario 9: Inlet Type 2, Orifice Flow, *HGL* below Soffit
10. Scenario 10: Inlet Type 2, Weir/Orifice Transition, *HGL* below Soffit
11. Scenario 11: Inlet Type 2, Orifice Flow, Ponded Depth, *HG*L below Soffit

12. Scenario 12: Inlet Type 2, Orifice Flow, *HGL* below Soffit, with Curb
13. Scenario 13: Inlet Type 3, Weir Flow, *HGL* below Soffit
14. Scenario 14: Inlet Type 3, Orifice Flow, *HGL* below Soffit
15. Scenario 15: Inlet Type 3, Weir/Orifice Transition, *HGL* below Soffit
16. Scenario 16: Inlet Type 3, Orifice Flow, Ponded Depth, *HG*L below Soffit
17. Scenario 17: Inlet Type 3, Weir/Orifice Transition, *HGL* below Soffit, with Curb
18. Scenario 18: Inlet Type 4, Rating Table, *WSEL* below soffit
19. Scenario 19: Inlet Type 4, Rating Table, *WSEL* above soffit, *HGL* below Soffit
20. Scenario 20: Inlet Type 4, Rating Table, *WSEL* above soffit, *HGL* above Soffit
21. Scenario 21: Inlet Type 1, Orifice Flow, *HGL* below Soffit, with Junction
22. Scenario 22: Inlet Type 5, Manhole Popped, Low Flow Out of Manhole
23. Scenario 23: Inlet Type 5, Manhole Popped, High Flow Out of Manhole
24. Scenario 24: Inlet Type 5, Manhole Popped, Weir Flow into Manhole
25. Scenario 25: Inlet Type 5, Manhole Popped, Orifice Flow into Manhole
26. Scenario 26: Inlet Type 5, Manhole Popped, Floodplain Grid *HGL* > Pipe *HGL*
27. Scenario 27: Inlet Type 5, Manhole Popped, Floodplain Grid *HGL* < Pipe *HGL*
28. Scenario 28: Inlet Type 5, Flow on Floodplain Does Not Allow Manhole to Pop
29. Scenario 29: Inlet Type 5, Manhole Popped, Storm Drain Surcharged, with Inlet Bypass Flow
30. Scenario 30: Outfall to Floodplain, with Flap Gate
31. Scenario 31: Outfall to Floodplain, Tailwater below Top of Pipe, above critical depth
32. Scenario 32: Outfall to Floodplain, Tailwater below Top of Pipe, below critical depth
33. Scenario 33: Outfall to *1D* Channel, Tailwater below Top of Pipe, above critical depth
34. Scenario 34: Outfall to *1D* Channel, Tailwater below Top of Pipe, below critical depth
35. Scenario 35: Outfall to Floodplain, Tailwater above Top of Pipe
36. Scenario 36: Outfall to *1D* Channel, Tailwater above Top of Pipe
37. Scenario 37: Outfall to Floodplain, with Lake Submergence (1_15cfs)
38. Scenario 38: Outfall to Floodplain, with Lake Submergence (2a)
39. Scenario 39: Outfall to Floodplain, with Lake Submergence (3)
40. Scenario 40: Outfall to *1D* Channel, with Lake Submergence
41. Scenario 41: Outfall to *1D* Channel, Tailwater below Soffit, above critical depth
42. Scenario 42: Outfall to Floodplain, Tailwater above Soffit
43. Scenario 43: Outfall to Floodplain, Tailwater above Top of Pipe

A standard storm drain with the following characteristics was used for these examples:

1. 30" concrete pipe, *n* = 0.020.
2. Pipe length = 400 feet
3. Pipe slope = 2%
4. Upstream and downstream channel with a base width = 10 ft, slope = 4%, *n* = 0.040
5. *SWMM* Flow routing: DYWAVE (Dynamic Wave)
6. *SWMM* Reporting step: 3 min
7. *SWMM* WET_STEP, DRY_STEP: 1 min
8. *SWMM* Routing Step: 1 sec
9. *SWMM* Allow ponding: Yes
10. *SWMM* Inertial damping: Partial
11. *SWMM* Variable Step: 0.5
12. *SWMM* Lengthening step: 0 sec
13. Curb Height = 0 feet, unless noted

The *FLO-2D* model primary control parameters for the entire storm drain model scenarios were:

1. *1D* channel component applied depending on the scenario.
2. No rainfall.
3. No infiltration.
4. No flow obstructions.
5. Inflow hydrograph. Refer to the various scenarios.
6. Depth variable *n*-values: off
7. Shallow *n*: off
8. Limiting *Froude number*: 0.00
9. Global ARF: 0
10. *TOL*: 0.001 feet
11. *1D* channel Courant Number: unless noted a value of 0.6 is used
12. Floodplain Grid Courant Number: unless noted a value of 0.6 is used
13. TIME_ACCEL: 0.10
14. Model simulation time: varies

4.2.7.2 Scenario 1: Inlet Type 1, Weir Flow, *HGL* below Soffit

Scenario

Test of the *FLO-2D/SWMM* approach for a storm drain connecting two floodplain grids with a Type 1 Inlet: Curb Opening Inlet at Grade under a weir flow condition. A plan view of the test model is shown on Figure 4.147 and a profile view on Figure 4.148. The inflow hydrograph is small enough that the *FLO-2D* water depth in the grid element that contains the inlet structure is less than the curb opening height, forcing use of the weir equation. The inlet has the following parameters:

1. Curb Opening Length = 3 feet
2. Curb Opening Height (h) = 0.5 feet
3. Weir Coefficient = 3.0
4. Curb Height = 0.0 feet

Figure 4.147 Plan View of Inlet Type 1, 2 & 3 Scenario Models

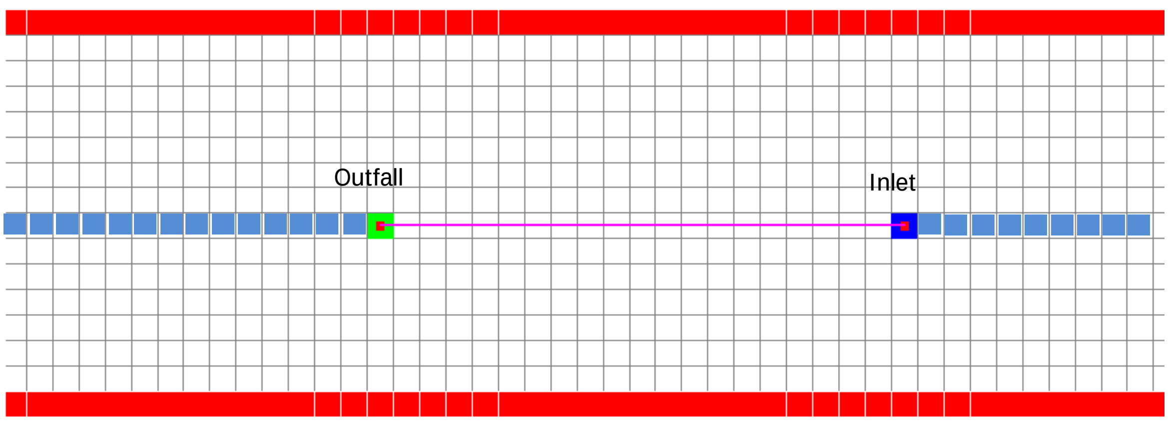
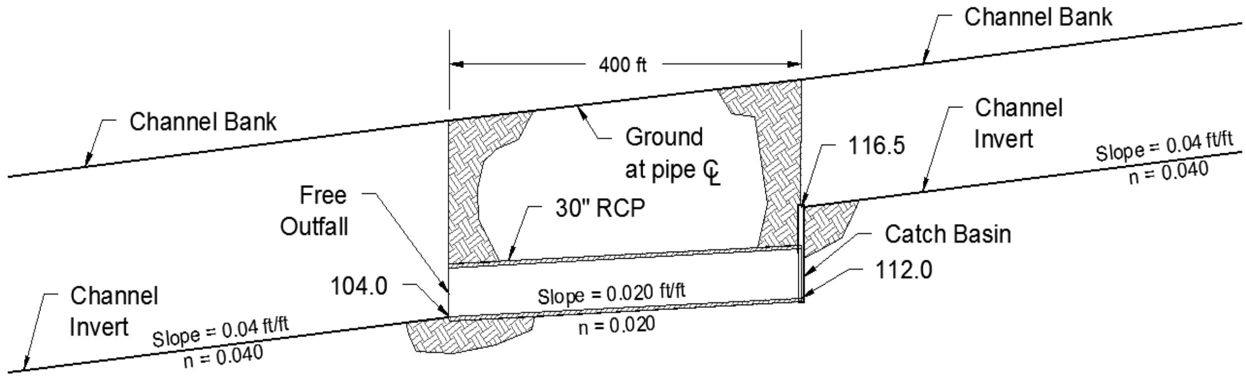


Figure 4.148 Profile View of Inlet Type 1, 2 & 3 Scenario Models



Benchmark

Direct comparison to the weir equation.

$$Q_w = CLH^m = 3.0 * 3.0 * 0.48^{1.5} = 3.0\ cfs$$

where:

Q_w = weir discharge at depth H, cfs

C = weir coefficient

L = curb opening length, feet

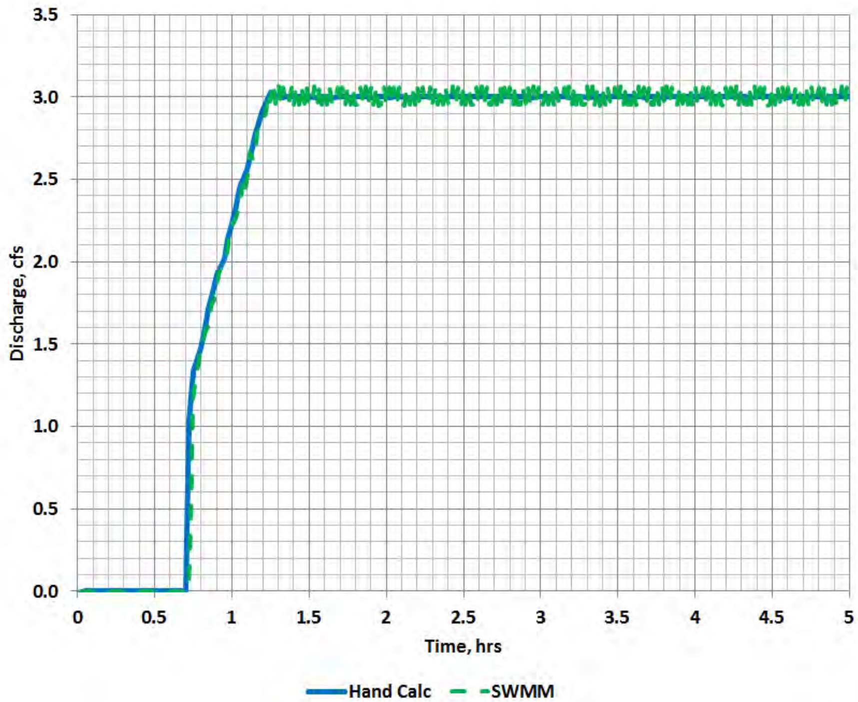
H = *FLO-2D* grid element water depth that contains the inlet structure, feet

m = 1.5 for a broad crested weir. This parameter is hardcoded in the *FLO-2D* program.

Discussion

The uniform flow rate in the storm drain as reported by *SWMM* is equal to 3.0 cfs, which matches the hand calculation using the weir equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. Comparison for the scenario is shown on Figure 4.149. This approach for modeling a storm drain with a Type 1 inlet under weir conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.149 Scenario 1: Inlet Type 1, Weir Flow



4.2.7.3 Scenario 2: Inlet Type 1, Orifice Flow, *HGL* below Soffit

<u>Scenario</u>

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 1 Inlet: Curb Opening Inlet at Grade under an orifice flow condition. A plan view of the test model is shown on Figure 4.147 and a profile view is shown on Figure 4.148. The inflow hydrograph has enough uniform discharge that the Orifice Equation Controls. This occurs when the *FLO-2D* grid element water depth that contains the inlet structure is more than 1.4 times the curb opening height. The inlet has the same configuration as Section 4.2.7.2.

<u>Benchmark</u>

Direct comparison to the orifice equation.

$$Q_o = C_d\, A \sqrt{2\, g\, H} = 0.67 * (3 * 0.5) * \sqrt{2 * 32.174 * 1.537} = 10.0\; cfs$$

where:

Q_o = orifice flow rate at depth H, cfs

C_d = discharge coefficient hardcoded in *FLO-2D* to 0.67

A = cross sectional orifice area, computed from curb opening length (L) and curb opening height (h), ft^2

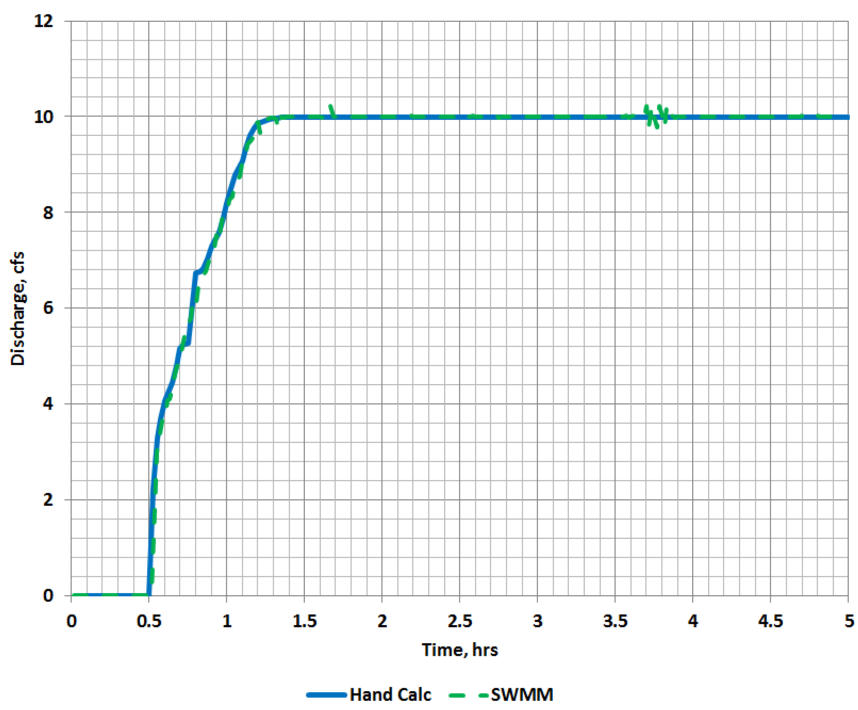
g = gravitational acceleration, ft/sec^2

H = *FLO-2D* grid element water depth that contains the inlet structure, feet

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 10.0 cfs, which matches the hand calculation of the orifice equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. Comparison for the scenario is shown on Figure 4.150. This approach for modeling a storm drain with a Type 1 inlet under orifice conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.150 Scenario 2: Inlet Type 1, Orifice Flow



4.2.7.4 Scenario 3: Inlet Type 1, Weir/Orifice Transition, *HGL* below Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 1 Inlet: Curb Opening Inlet at Grade that has a transition from weir to orifice flow condition. A plan view of the model is shown on Figure 4.147 and a profile view on Figure 4.148. The inflow hydrograph has a discharge designed to force flow in the transition zone where:

If $h \leq H < 1.4\ h$, where h is the curb opening height, ft, and H is the flow depth on the *FLO-2D* grid element, ft

If $Q_w \leq Q_o$ then the discharge = Q_w (weir equation controls)

If $Q_w > Q_o$ then the discharge = Q_o (orifice equation controls)

Benchmark

Direct comparison to the smallest value from the solution of either the weir or orifice equation:

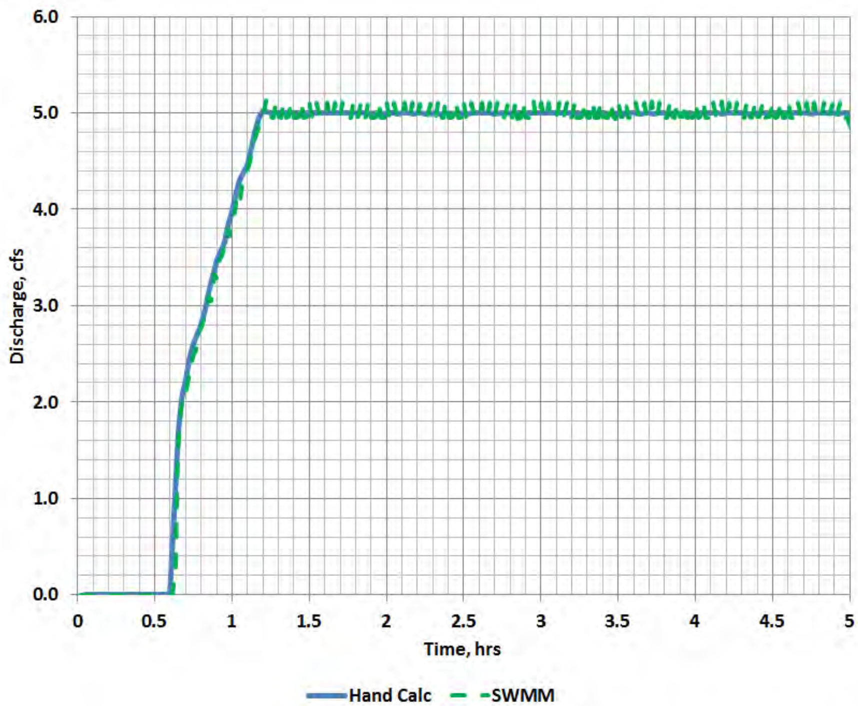
$$Q_w = CLH^m = 3.0 * 3.0 * 0.676^{1.5} = 5.0\ cfs - This\ equation\ controls.$$

$$Q_o = C_d\ A\ \sqrt{2\ g\ H} = 0.67 * (3.0 * 0.5) * \sqrt{2 * 32.2 * 0.676} = 6.6\ cfs$$

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 5.0 cfs, which matches the hand calculation using the weir equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. A comparison for the scenario is shown on Figure 4.151. This approach for modeling a storm drain with a Type 1 inlet under transition conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.151 Scenario 3: Inlet Type 1, Transition Flow



4.2.7.5 Scenario 4: Inlet Type 1, Orifice Flow, Ponded Depth, *HGL* below Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 1 Inlet: Curb Opening Inlet at Grade that has ponding on the inlet. A plan view of the test model is shown on Figure 4.147 and a profile view on Figure 4.148. The inflow hydrograph has enough uniform discharge that the Orifice Equation Controls and there is ponding on the inlet, but not enough that there is pressure flow in the storm drain. The inlet has the same parameters as Section 4.2.7.2.

Benchmark

Direct comparison to the smallest value from the solution of either the weir or orifice equation at time 0.05 hrs (close to the peak discharge):

$$Q_w = CLH^m = 3.0 * 3.0 * 13.931^{1.5} = 468.0\, cfs$$

$$Q_o = C_d\, A \sqrt{2\, g\, H} = 0.67 * (3.0 * 0.5) * \sqrt{2 * 32.2 * 13.931} = 30.1\, cfs - This\ equation\ contols$$

The depth in the above equation was obtained from the TIMDEP.OUT file at grid 746 (grid at the *SWMM* inlet).

Discussion

The water surface elevation in the storm drain (obtained from the *SWMM*.RPT file and labeled as *SWMM* Inlet) and on the floodplain grid (obtained from the TIMDEP.OUT file and labeled as TIMDEP) is shown on Figure 4.152. A comparison to a hand calculation of Type 1 inlet equations (either orifice or weir equations) matched the results of the model and this is shown on Figure 4.153. At time 0.05 hrs the *SWMM*.RPT file reports an inflow discharge of 30.4 cfs, which is similar to the above hand calculation of 30.1 cfs. This approach for modeling a storm drain with a Type 1 inlet under inlet pressure conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.152 Scenario 4: Inlet Type 1, Water Surface Elevation

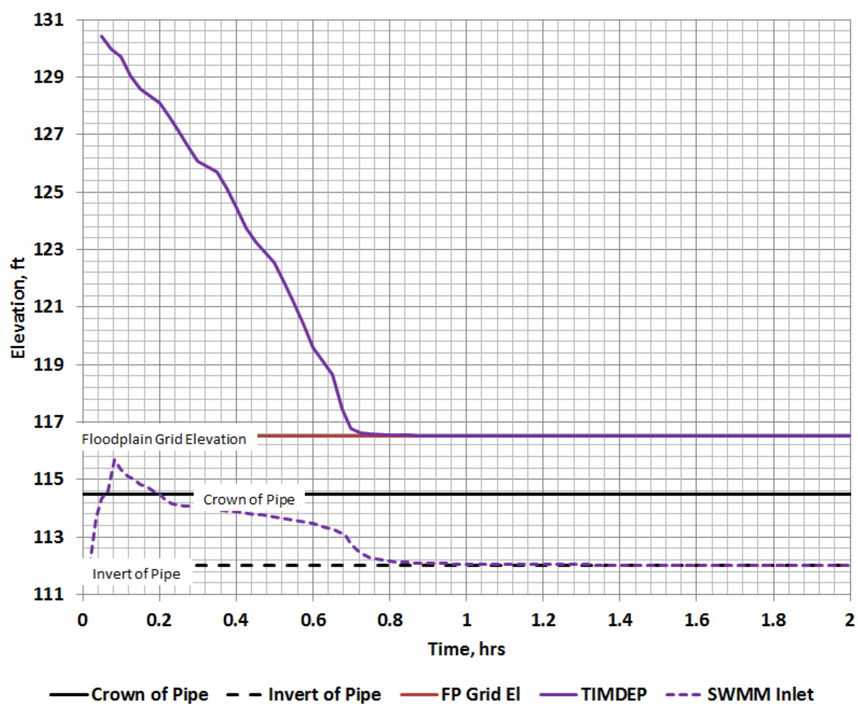
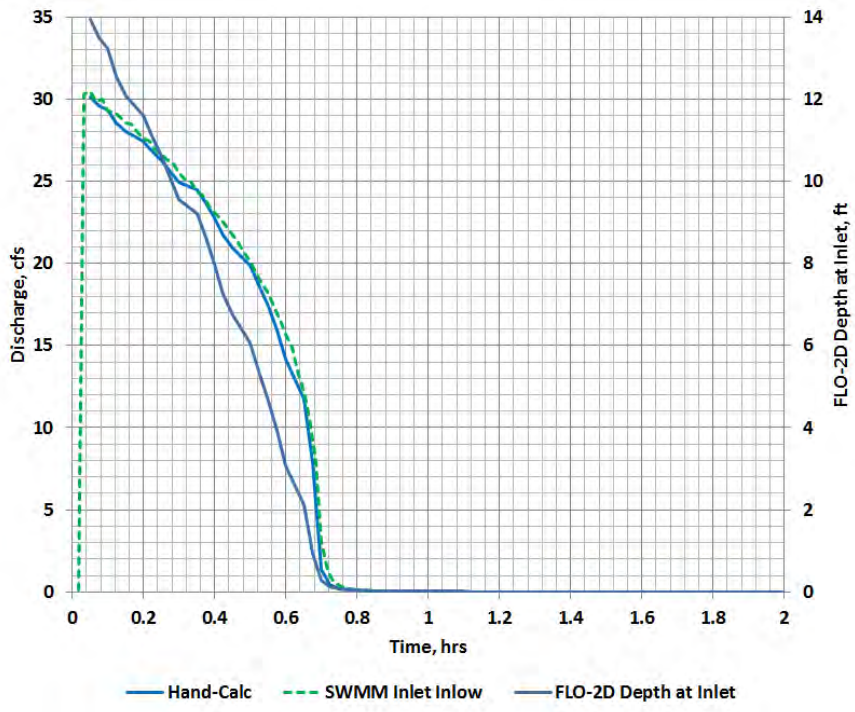


Figure 4.153 Scenario 4: Inlet Type 1, Inlet Pressure



4.2.7.6 Scenario 5: Inlet Type 1, Orifice Flow, Pipe Surcharged, *WSEL* > *HGL* > Inlet

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 1 Inlet: Curb Opening Inlet at Grade that has pressure flow in the storm drain. The starting hydraulic grade line in the pipe is above the crown of the pipe and above the floodplain grid elevation. The plan view of the test model is shown on Figure 4.147 and a profile view on Figure 4.148. The inflow hydrograph has enough uniform discharge that the Orifice Equation Controls and there is pressure flow in the storm drain. The inlet has the same parameters as Section 4.2.7.2. The inlet has the following parameters:

1. Curb Opening Length (L) = 6 feet
2. Curb Opening Height (h) = 1.0 feet
3. Weir Coefficient (C) = 3.0
4. Curb Height = 0.0 feet

Benchmark

Direct comparison to the smallest value from the solution of either the weir or orifice equation:

$$Q_w = CLH^m = 3.0 * 6.0 * 25.958^{1.5} = 2380.6\ cfs$$

$$Q_o = C_d\, A \sqrt{2\ g\ H} = 0.67 * (6.0 * 1.0) * \sqrt{2 * 32.2 * 25.958}$$
$$= 164.3\ cfs - This\ equation\ controls.$$

The depth in the above equation was obtained from the TIMDEP.OUT file at grid 746 (grid at the *SWMM* inlet).

Discussion

The water surface elevation in the storm drain (obtained from the *SWMM*.RPT file and labeled as *SWMM* Inlet) and on the floodplain grid (obtained from the TIMDEP.OUT file and labeled as TIMDEP) is shown on Figure 4.154. A comparison to a hand calculation of Type 1 inlet equations (either orifice or weir equations) is shown on Figure 4.155. In this scenario the hand calculation of the Type 1 inlet equations is higher than the results of the model, because the pipe capacity controls the discharge through the pipe rather than the inlet capacity. This approach for modeling a storm drain with a Type 1 inlet under inlet pressure conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.154 Scenario 5: Inlet Type 1, Water Surface Elevation

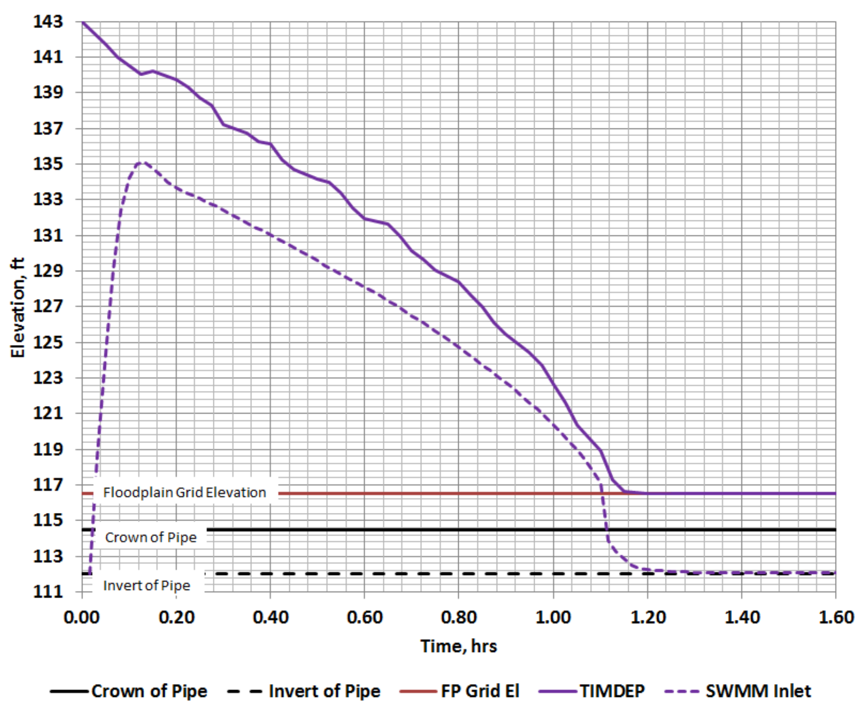
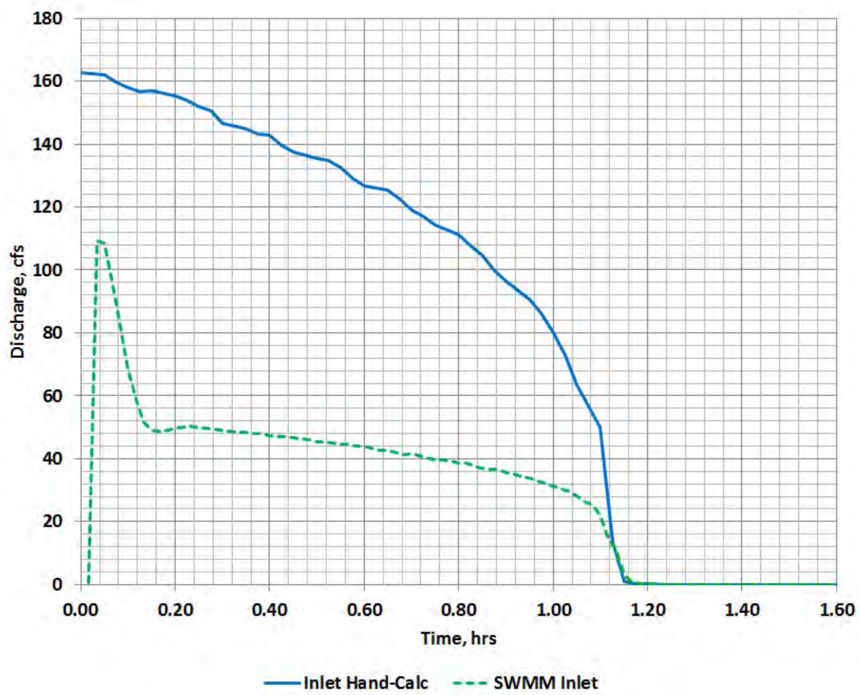


Figure 4.155 Scenario 5: Inlet Type 1, Pressure Pipe



4.2.7.7 Scenario 6: Inlet Type 1, Scenario 5 with Bypass Flow

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 1 Inlet: Curb Opening Inlet at Grade with flows large enough that some continues past the inlet into a secondary channel. The purpose of this test is to verify that the amount of flow entering the storm drain plus the bypass flow to the secondary channel is equivalent to the total flow entering the model. Also, the amount of flow entering the inlet is great enough that the storm drain is under pressure flow. The plan view of the test model is shown on Figure 4.156 and the profile view on Figure 4.157.

Figure 4.156 Scenario 6: Plan View

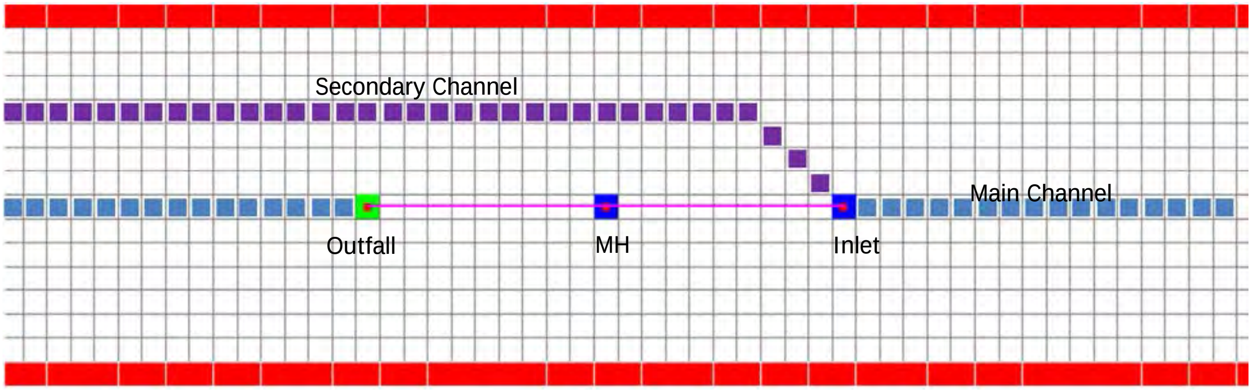
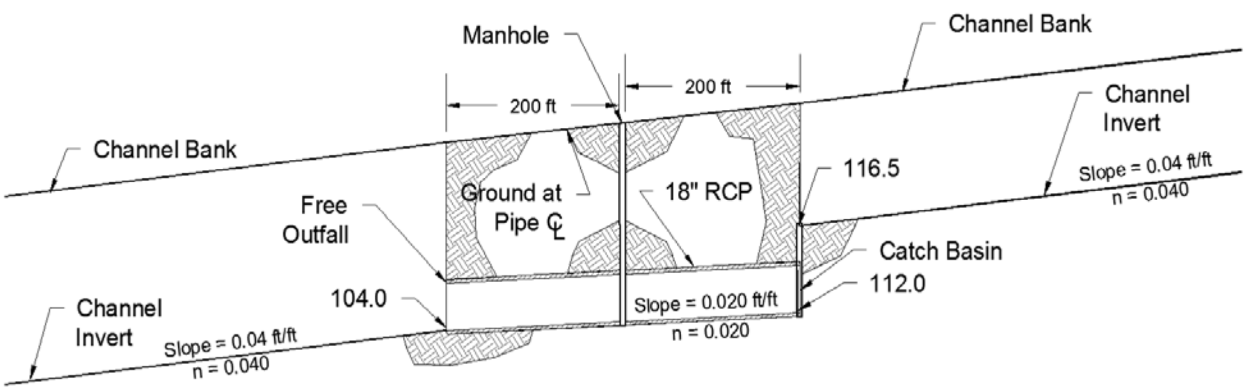


Figure 4.157 Scenario 6: Profile View



Benchmark

The check for this scenario is that the sum of the flow entering the storm drain plus the flow continuing down the secondary channel equals the total flow entering the model.

Discussion

The flow entering the storm drain was obtained from the *SWMM*.RPT file for the inlet node. The flow continuing past the inlet was obtained from the HYCROSS.OUT file. These results are shown on Figure 4.158 along with the summation and the total flow entering the model. The surcharge on the storm drain is shown on Figure 4.159. The offset in time between the inflow and the total flow entering the inlet and the bypass flow is due to travel time. This approach for modeling a storm drain with a Type 1 inlet under inlet pressure conditions with flow past the inlet functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.158 Scenario 6: Type 1 with Bypass Flow

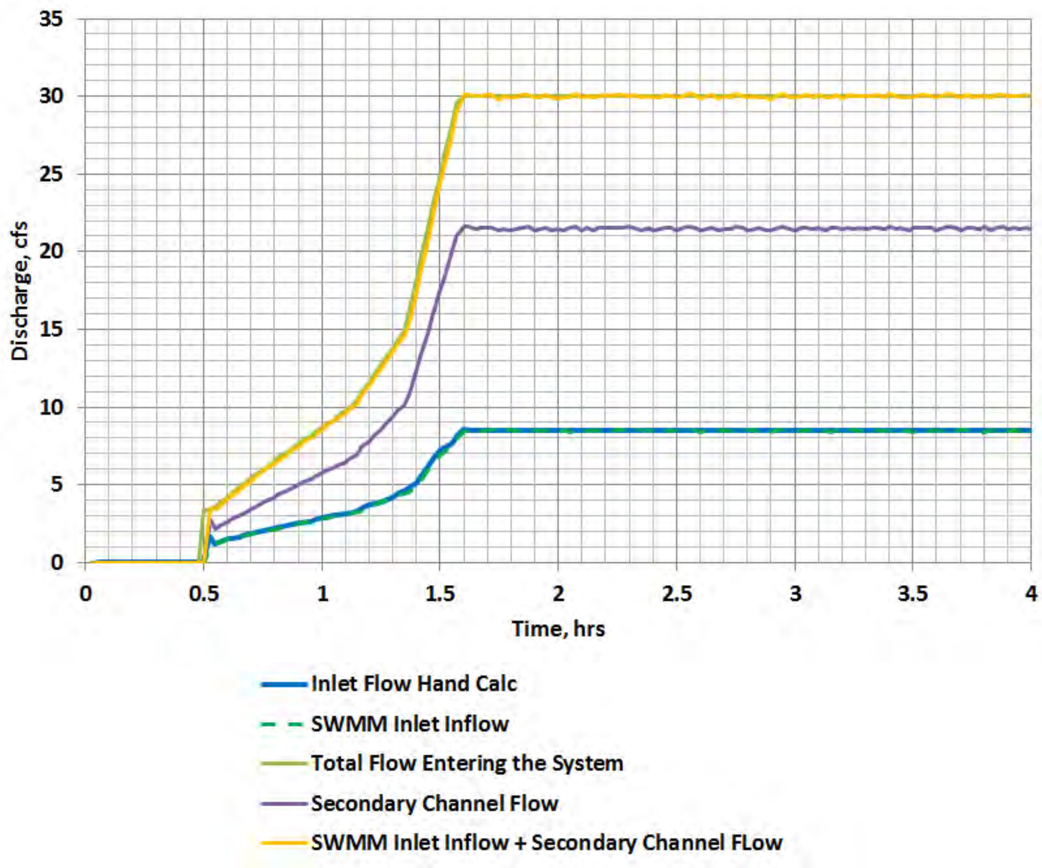
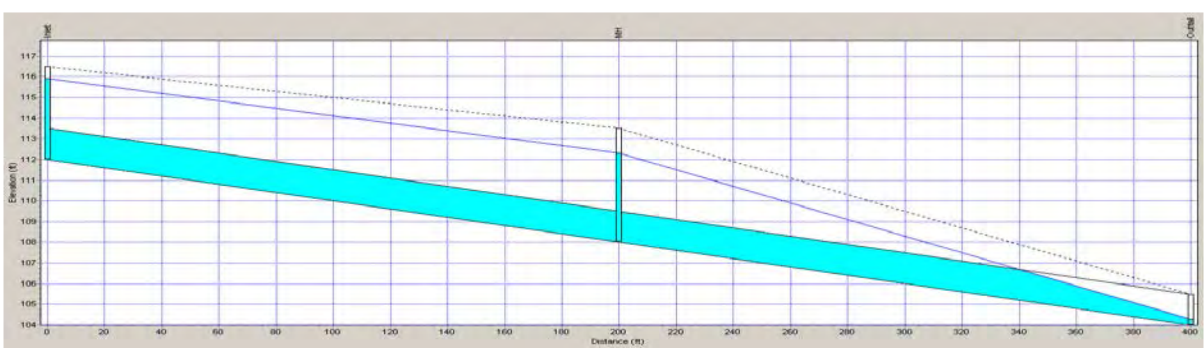


Figure 4.159 Scenario 6: Storm Drain Profile



4.2.7.8 Scenario 7: Inlet Type 1, Weir Flow, *HGL* below Soffit, with Curb

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 1 Inlet: Curb Opening Inlet at Grade that has weir flow and a curb. Setting a curb height will allow a higher depth on the storm drain inlet than occurs from a flat grid element and therefore an increase of flow into the storm drain system. When a curb height is set, the storage volume available on the grid is also increased using an assumed 2% cross slope. Refer the *FLO-2D Storm Drain Manual* (FLO-2D Software, Inc., 2015f) for more information. The plan view of the test model is shown on Figure 4.147 and the profile view on Figure 4.148. The inflow hydrograph is small enough that the *FLO-2D* water depth in the grid element that contains the inlet structure is less than the curb opening height, forcing use of the weir equation. The inlet has the following parameters:

1. Curb Opening Length (L) = 3 feet
2. Curb Opening Height (h) = 0.5 feet
3. Weir Coefficient (C) = 3.0
4. Curb Height = 0.5 feet
5. Flow depth on FLO-2D grid (H) = 0.426 ft
6. Floodplain Grid Courant Number = 0.2

Benchmark

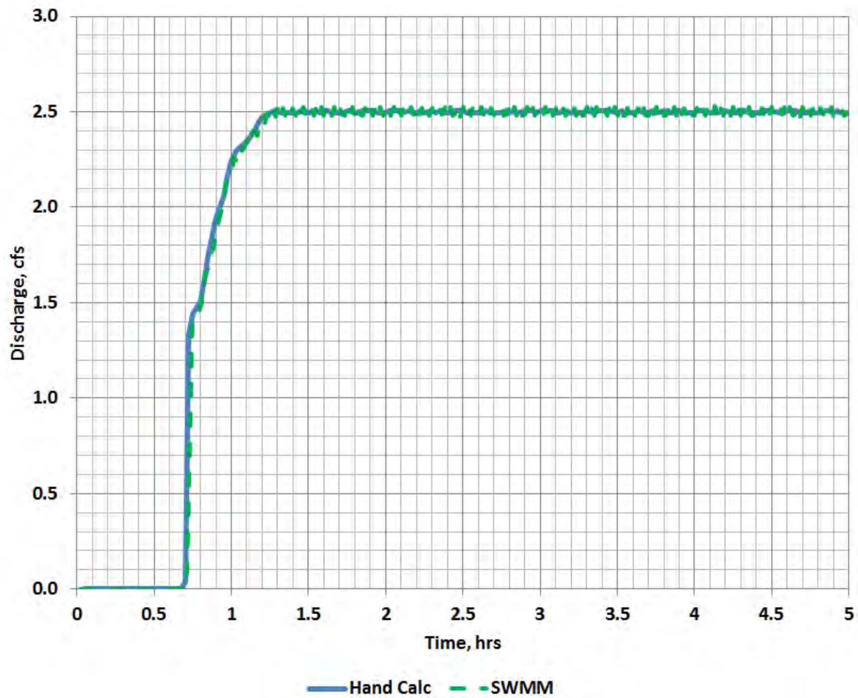
Direct comparison to the weir equation:

$$Q_w = CLH^m = 3.0 * 3.0 * 0.426^{1.5} = 2.5\ cfs$$

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 2.5 cfs, which matches the hand calculation of the weir equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. A comparison for the scenario is shown on Figure 4.160. This approach for modeling a storm drain with a Type 1 inlet under weir conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.160 Scenario 7: Inlet Type 1, Weir Flow with a Curb



4.2.7.9 Scenario 8: Inlet Type 2, Weir Flow, *HGL* below Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 2 Inlet: Curb Opening Inlet with Sag that has weir flow. The plan view of the test model is shown on Figure 4.147 and the profile view on Figure 4.148. The inflow hydrograph is small enough that the *FLO-2D* water depth in the grid element that contains the inlet structure is less than the curb opening height, forcing the use of the weir equation. The curb has the following parameters:

1. Curb Opening Length = 3.0 feet
2. Curb Sag Width = 0.8 feet
3. Curb Opening Height = 0.5 feet
4. Weir Coefficient = 2.3
5. Curb Height = 0.0 feet

Benchmark

Direct comparison to the weir equation.

$$Q_w = C\,(L + 1.8W)H^m = 2.3\,(3.0 + 1.8 * 0.8) * 0.442^{1.5} = 3.0\,cfs$$

where:

Q_w = weir flow rate at depth H, cfs

C = weir coefficient

L = curb opening length, feet

W = curb sag width, feet

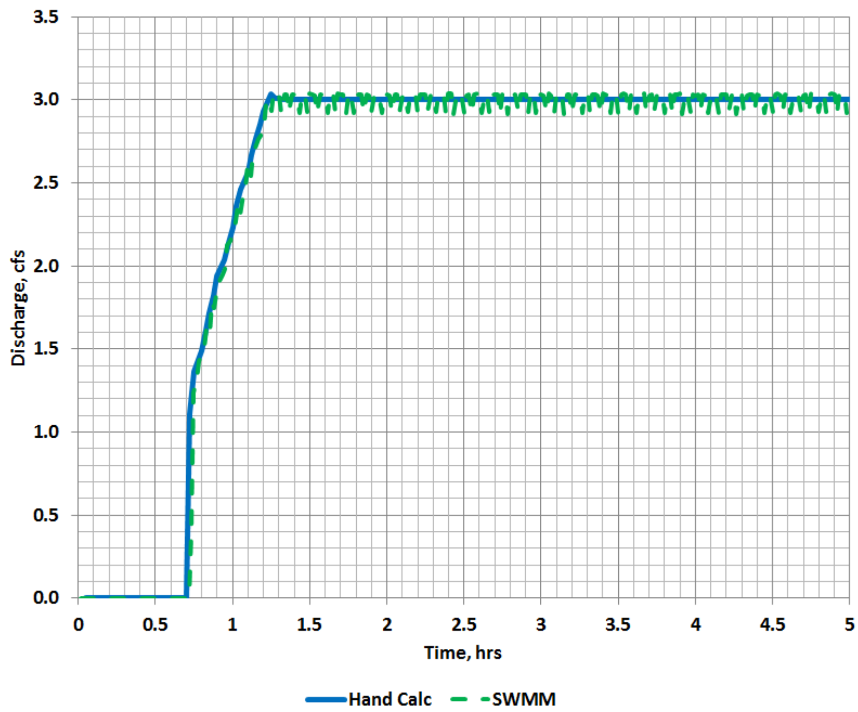
H = *FLO-2D* grid element water depth that contains the inlet structure, feet

m = 1.5 exponent for a horizontal weir, hardcoded

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 3.0 cfs, which matches the hand calculation of the weir equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. Comparison for the scenario is shown on Figure 4.161. This approach for modeling a storm drain with a Type 2 inlet under weir conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.161 Scenario 8: Inlet Type 2, Weir Flow



4.2.7.10 Scenario 9: Inlet Type 2, Orifice Flow, *HGL* below Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 2 Inlet: Curb Opening Inlet with Sag that has orifice flow. The plan view of the test model is shown on Figure 4.147 and the profile view on Figure 4.148. The inflow hydrograph has a discharge great enough that the orifice equation controls. This occurs when the *FLO-2D* grid element water depth that contains the inlet structure is more than 1.4 times the curb opening height. The curb has the same parameters as Section 4.2.7.9.

Benchmark

Direct comparison to the solution of the following orifice equations:

$$If\ h \geq H\ then\ Q_o = C_d\ A\ \sqrt{2\ g\ H}$$

$$If\ h < H\ then\ Q_o = C_d\ A\ \sqrt{2\ g\ (H - \frac{h}{2})} = 0.67 * (3 * 0.5) * \sqrt{2 * 32.174 * \left(1.787 - \frac{0.5}{2}\right)} = 10.0\text{ cfs}$$

where:

Q_o = orifice flow rate at depth H, cfs

C_d = discharge coefficient hardcoded to 0.67

$A = L*h$, cross sectional orifice area, computed from curb opening length (L) and curb opening height (h), ft^2

g = gravitational acceleration, ft/sec^2

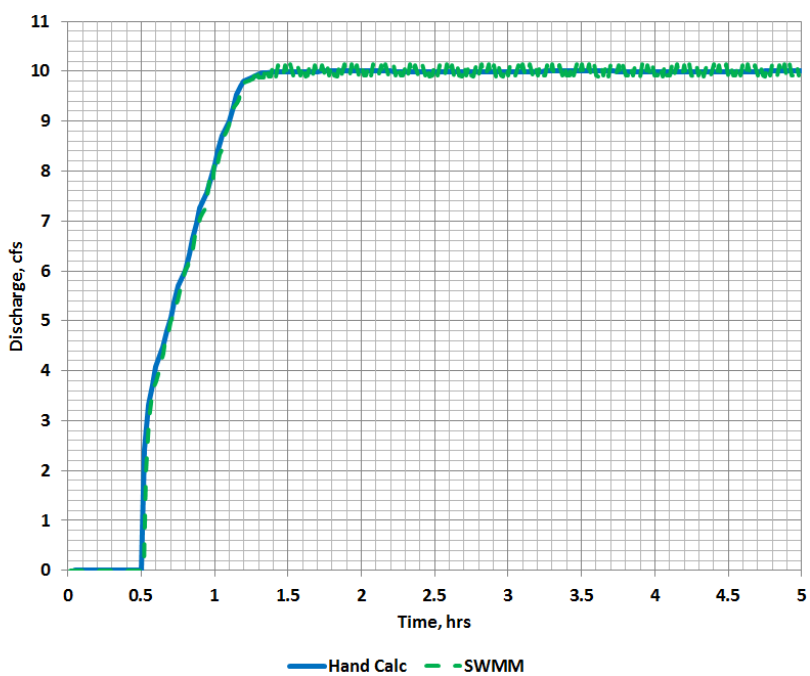
H = *FLO-2D* grid element water depth that contains the inlet structure, feet

h = height of the orifice (curb opening height), ft

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 10.0 cfs, which matches the hand calculation of the orifice equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. A comparison for the scenario is shown on Figure 4.162. This approach for modeling a storm drain with a Type 2 inlet under orifice conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.162 Scenario 9: Inlet Type 2, Orifice Flow



4.2.7.11 Scenario 10: Inlet Type 2, Weir/Orifice Transition, *HGL* below Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 2 Inlet: Curb Opening Inlet with Sag that has flow transitioning from weir to orifice flow. A plan view of the test model is shown on Figure 4.147 and a profile view on Figure 4.148. The inflow hydrograph has a discharge designed to force depths to be in the transition zone:

If $h \leq H < 1.4\ h$ – Transition Zone

If $Q_w \leq Q_o$ then the discharge = Q_w (weir equation controls)

If $Q_w > Q_o$ then the discharge = Q_o (orifice equations control)

Benchmark

Direct comparison to the smallest value from the solution of either the weir or orifice equations:

Weir Equation:

$$Q_w = C\ (L + 1.8W)H^m = 2.3\ (3.0 + 1.8 * 0.8) * 0.634^{1.5} = 5.2\ cfs$$

Orifice Equations:

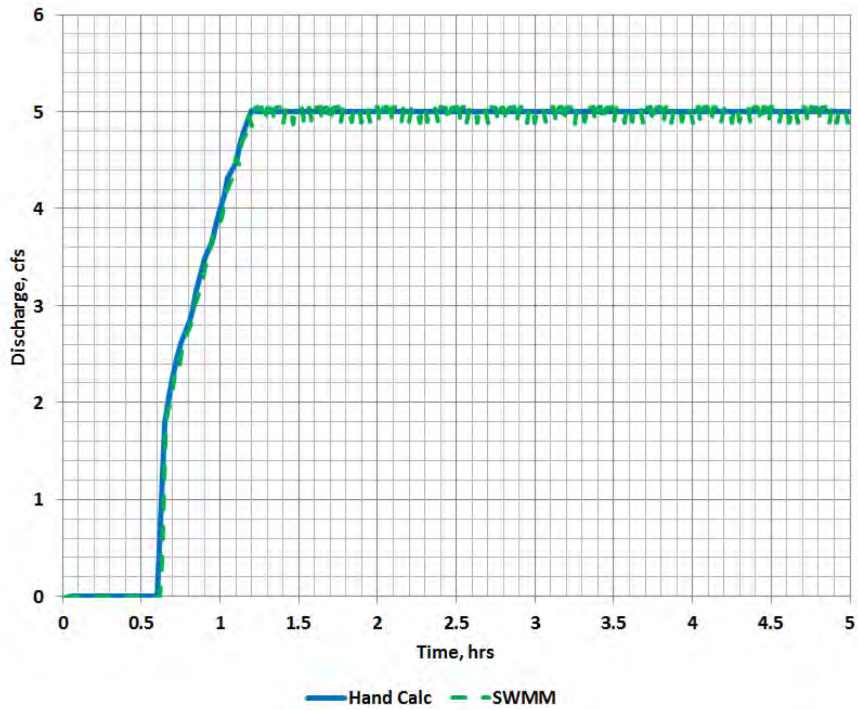
$$If\ h \geq H\ then\ Q_o = C_d\ A\ \sqrt{2\ g\ H}$$

$$If\ h < H\ then\ Q_o = C_d\ A\ \sqrt{2\ g\ (H - \frac{h}{2})} = 0.67 * (3 * 0.5)\sqrt{2 * 32.174 * \left(0.634 - \frac{0.5}{2}\right)} = 5.0\ \text{cfs}$$

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 5.0 cfs, which matches the hand calculation of the orifice equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. Comparison for the scenario is shown on Figure 4.163. This approach for modeling a storm drain with a Type 2 inlet under transition conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.163 Scenario 10: Inlet Type 2, Transition Flow



4.2.7.12 Scenario 11: Orifice Flow, Ponded Depth, *HG*L below Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 2 Inlet: Curb Opening Inlet with Sag that has ponding on the inlet. The plan view of the test model is shown on Figure 4.147 and the profile view on Figure 4.148. The inlet has the same parameters as defined in Section 4.2.7.9.

Benchmark

Direct comparison to the smallest value from the solution of either the weir or orifice equation at time 0.05 hrs (near the peak):

Weir Equation:

$$Q_w = C\ (L + 1.8W)H^m = 2.3\ (3.0 + 1.8 * 0.8) * 15.945^{1.5} = 650.2\ cfs$$

Orifice Equations:

$$If\ h \geq H\ then\ Q_o = C_d\ A\ \sqrt{2\ g\ H}$$

$$If\ h < H\ then\ Q_o = C_d\ A\ \sqrt{2\ g\ (H - \frac{h}{2})} = 0.67 * (3 * 0.5)\sqrt{2 * 32.174 * \left(15.945 - \frac{0.5}{2}\right)} = 31.9 \text{ cfs}$$

Discussion

The water surface elevation in the storm drain and on the floodplain grid is shown on Figure 4.164. A comparison to a hand calculation of Type 2 inlet equations (either orifice or weir equations) matched the results of the model and this is shown on Figure 4.165. At time 0.05 hrs the *SWMM*.RPT file reports an inflow discharge of 32.2 cfs, which is comparable to the above hand calculation of 31.9 cfs. This approach for modeling a storm drain with a Type 2 inlet under inlet pressure conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.164 Scenario 11: Inlet Type 2, Water Surface Elevation

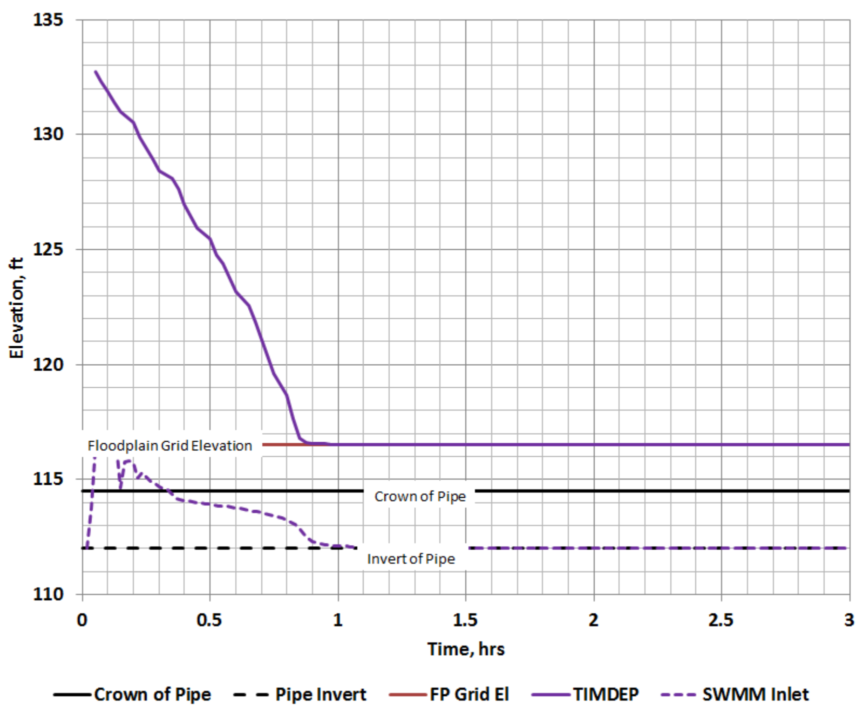
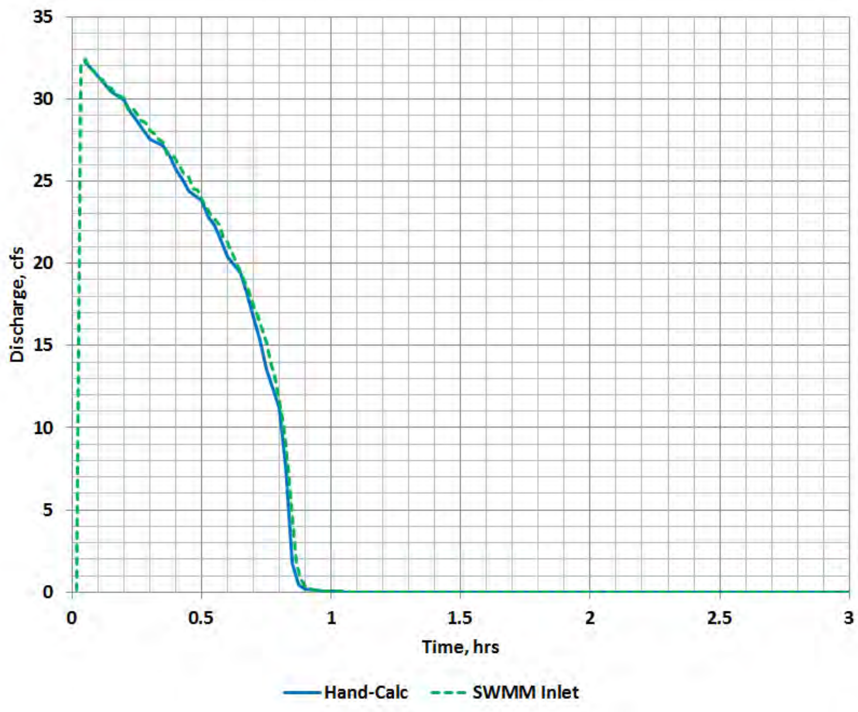


Figure 4.165 Scenario 11: Inlet Type 2, Inlet Pressure



4.2.7.13 Scenario 12: Inlet Type 2, Orifice Flow, *HGL* below Soffit, with Curb

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 2 Inlet: Curb Opening Inlet with Sag that has orifice flow and a curb. The inflow hydrograph has enough uniform discharge that the Orifice Equation Controls. This occurs when the *FLO-2D* grid element water depth that contains the inlet structure is more than 1.4 times the curb opening height. This scenario has the same physical parameters as Section 4.2.7.9 except a curb height is set.

Benchmark

Direct comparison to the solution of the following orifice equations:

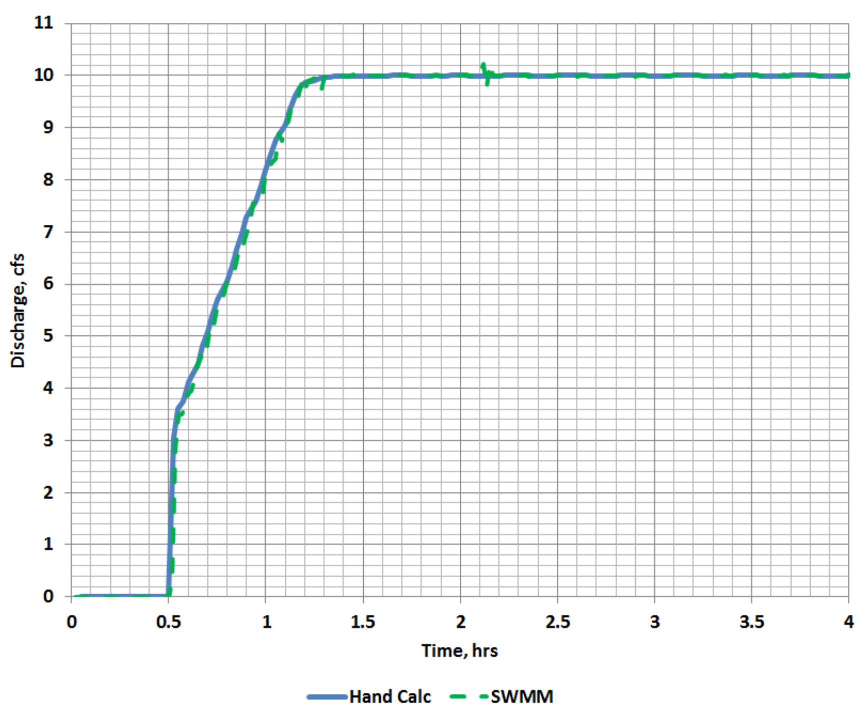
$$If\ h \geq H\ then\ Q_o = C_d\ A\sqrt{2\ g\ H} = 0.67 * (3 * 0.5) * \sqrt{2 * 32.2 * 1.8} = 10.0\ \text{cfs}$$

$$If\ h < H\ then\ Q_o = C_d\ A\sqrt{2\ g\ (H - \frac{h}{2})}$$

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 10.0 cfs, which matches the hand calculation of the orifice equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. A comparison for the scenario is shown on Figure 4.166. This approach for modeling a storm drain with a Type 2 inlet under orifice conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.166 Scenario 12: Inlet Type 2, Orifice Flow, with a Curb



4.2.7.14 Scenario 13: Inlet Type 3, Weir Flow, *HGL* below Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 3 Inlet: Grate (Gutter) Inlet with/without Sag that has weir flow. The plan view of the test model is shown on Figure 4.147 and the profile is shown on Figure 4.148. The inflow hydrograph is small enough that the *FLO-2D* water depth in the grid element that contains the inlet structure is less than 0.75 feet deep, forcing the use of the weir equation. The inlet has the following parameters:

1. Weir Coefficient = 3.0

2. Grate Perimeter = 7.33 feet
3. Grate Open Area = 1.22 feet^2
4. Grate Sag Height = 0.0 feet
5. Curb Height = 0.0 feet

Benchmark

Direct comparison to the smaller of the two weir equations:

$$If\ H\ \leq h\ then: Q_w = CPH^m$$

$$If\ H > h\ then: Q_w = CP\left(H + \frac{h}{2}\right)^m = 3.0 * 7.33 * \left(0.265 + \frac{0}{2}\right)^{1.5} = 3.0 \text{ cfs}$$

Discussion

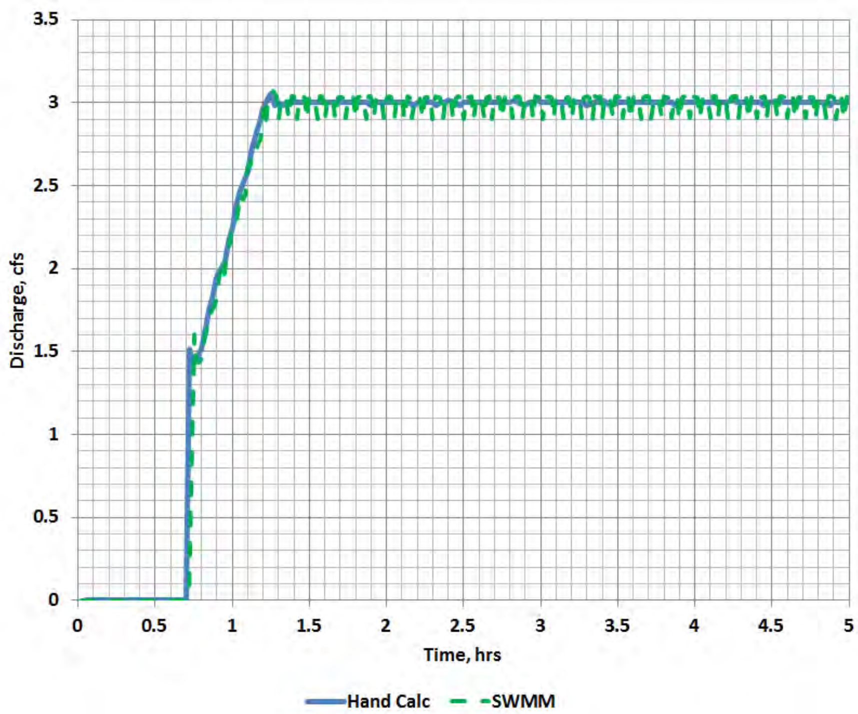
The flow rate in the storm drain as reported by *SWMM* is equal to 3.0 cfs, which matches the hand calculation using the weir equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. A comparison for the scenario is shown on Figure 4.167. This approach for modeling a storm drain with a Type 3 inlet under weir conditions functions as expected. This is acceptable for *FCDMC* purposes.

4.2.7.15 Scenario 14: Inlet Type 3, Orifice Flow, *HGL* below Soffit

Scenario

Test of the *FLO-2D/SWMM* approach for a storm drain connecting two floodplain grids with a Type 3 Inlet: Grate (Gutter) Inlet with/without Sag that has weir flow. The plan view of the test model is shown on Figure 4.147 and the profile view on Figure 4.148. The inflow hydrograph is great enough that the orifice equation controls. This occurs when the *FLO-2D* grid element water depth that contains the inlet structure is more than 1.8 feet deep. The inlet has the same parameters as Section 4.2.7.14.

Figure 4.167 Scenario 13: Inlet Type 3, Weir Flow



Benchmark

Direct comparison to the smaller of the Orifice Equations:

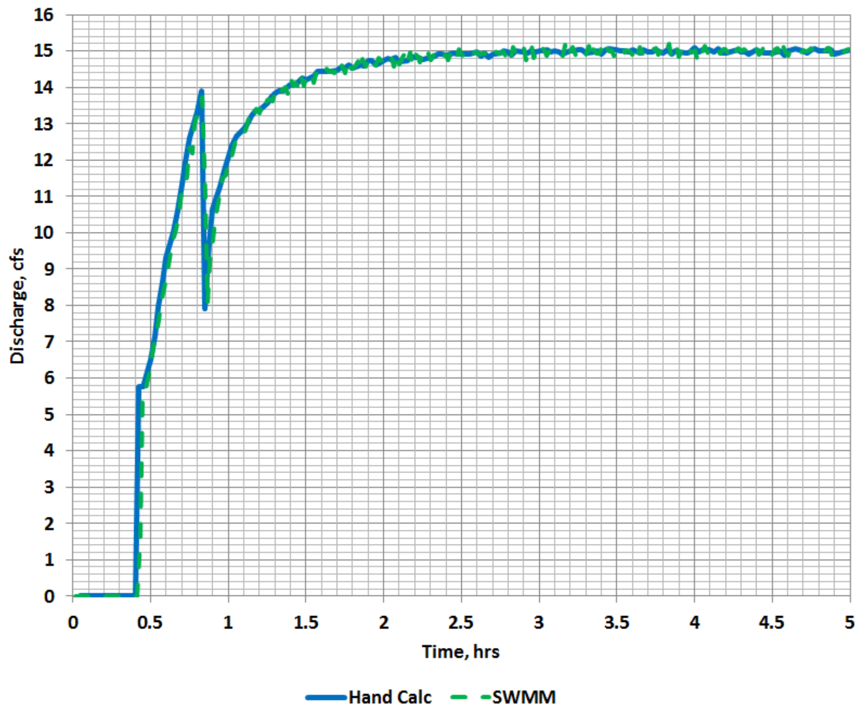
$$If\ H\ \leq h\ then: Q_o =\ C_d\ A\sqrt{2\ g\ H}$$

$$If\ H\ > h\ then: Q_o =\ \ C_d\ A\ \sqrt{2\ g\ \left(H + \frac{h}{2}\right)} = 0.67 * 1.22 *\ \sqrt{2 * 32.2\ \left(5.201 + \frac{0}{2}\right)} =\ 15.0\ \text{cfs}$$

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 15.0 cfs, which matches the hand calculation of the orifice equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. A comparison for the scenario is shown on Figure 4.168. Note that the weir equation controls the rising limb until 14 cfs and the depth reaches the transition zone. Then orifice flow is assumed resulting in the spike in the inflow hydrograph. This approach for modeling a storm drain with a Type 3 inlet under orifice conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.168 Scenario 14: Inlet Type 3, Orifice Flow



4.2.7.16 Scenario 15: Inlet Type 3, Weir/Orifice Transition, *HGL* below Soffit

Scenario

Test of the *FLO-2D/SWMM* approach for a storm drain connecting two floodplain grids with a Type 3 Inlet: Grate (Gutter) Inlet with/without Sag that has flow transitioning between weir and orifice flow. A plan view of test model is shown on Figure 4.147 and a profile view on Figure 4.148. The inlet has the following parameters:

1. Weir Coefficient = 3.0
2. Grate Perimeter = 6.28 feet
3. Grate Open Area = 3.14 feet^2
4. Grate Sag Height = 0.0 feet
5. Curb Height = 0.0 feet

The inflow hydrograph has a discharge designed to force the transition zone where:

If $0.75 \leq H < 1.8$

If $Q_w \leq Q_o$ then the discharge = Q_w (weir equation controls)

If $Q_w > Q_o$ then the discharge = Q_o (orifice equation controls)

Benchmark

Direct comparison to the smallest value from the solution of either the weir or orifice equations:

Weir Equations:

$$If\ H\ \leq h\ then: Q_w = CPH^m$$

$$If\ H > h\ then: Q_w = CP\left(H + \frac{h}{2}\right)^m = 3.0 * 6.28 * \left(1.403 + \frac{0}{2}\right)^{1.5} = \text{ 31.3 cfs}$$

Orifice Equations:

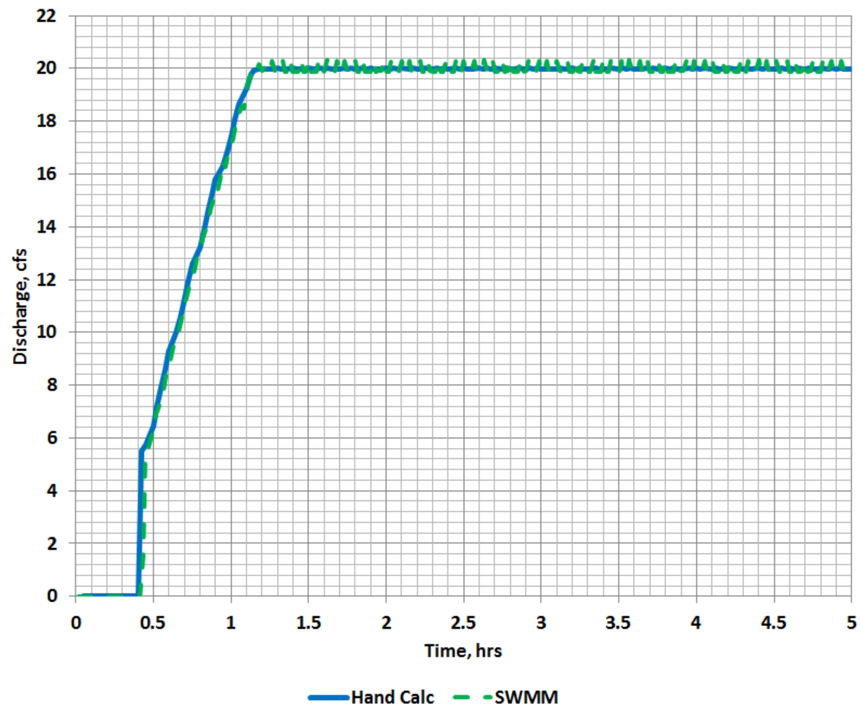
$$If\ H\ \leq h\ then: Q_o = C_d\ A\sqrt{2\ g\ H}$$

$$If\ H\ > h\ then: Q_o = C_d\ A\sqrt{2\ g\ \left(H + \frac{h}{2}\right)} = 0.67 * 3.14\sqrt{2 * 32.2\ \left(1.403 + \frac{0}{2}\right)} = \text{ 20.0 cfs}$$

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 20.0 cfs, which matches the hand calculation of the orifice equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. Comparison for the scenario is shown on Figure 4.169. This approach for modeling a storm drain with a Type 3 inlet under transition conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.169 Scenario 15: Inlet Type 3, Transition Flow



4.2.7.17 Scenario 16: Inlet Type 3, Orifice Flow, Ponded Depth, *HG*L below Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 3 Inlet: Grate (Gutter) Inlet with/without Sag that has ponding on the inlet. A plan view of the test model is shown on Figure 4.147 and a profile view on Figure 4.148 shows the profile view. The inlet has the same parameters as Section 4.2.7.14.

Benchmark

Direct comparison to the smallest value from the solution of either the weir or orifice equation at time 0.05 hrs (near the peak):

Weir Equations:

$$If\ H\ \leq h\ then\colon Q_w = CPH^m$$

$$If\ H > h\ then\colon Q_w = CP\left(H + \frac{h}{2}\right)^m = 3.0 * 7.33 * \left(21.087 + \frac{0}{2}\right)^{1.5} = 2129.35 \text{ cfs}$$

Orifice Equations:

$$If\ H\ \leq h\ then\colon Q_o = C_d\, A\sqrt{2\, g\, H}$$

$$If\ H\ > h\ then\colon Q_o = C_d\, A\sqrt{2\, g\left(H + \frac{h}{2}\right)} = 0.67 * 3.14\sqrt{2 * 32.2\left(21.087 + \frac{0}{2}\right)} = 30.1 \text{ cfs}$$

Discussion

The water surface elevation in the storm drain and on the floodplain grid is shown on Figure 4.170. A comparison to a hand calculation of Type 3 inlet equations (either orifice or weir equations) matched the results of the model and this is shown on Figure 4.171. At time 0.05 hrs the *SWMM*.RPT file reports an inflow discharge of 30.3 cfs, which is similar to the above hand calculation of 30.1 cfs. This approach for modeling a storm drain with a Type 3 inlet under inlet pressure conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.170 Scenario 16: Inlet Type 3, Water Surface Elevation

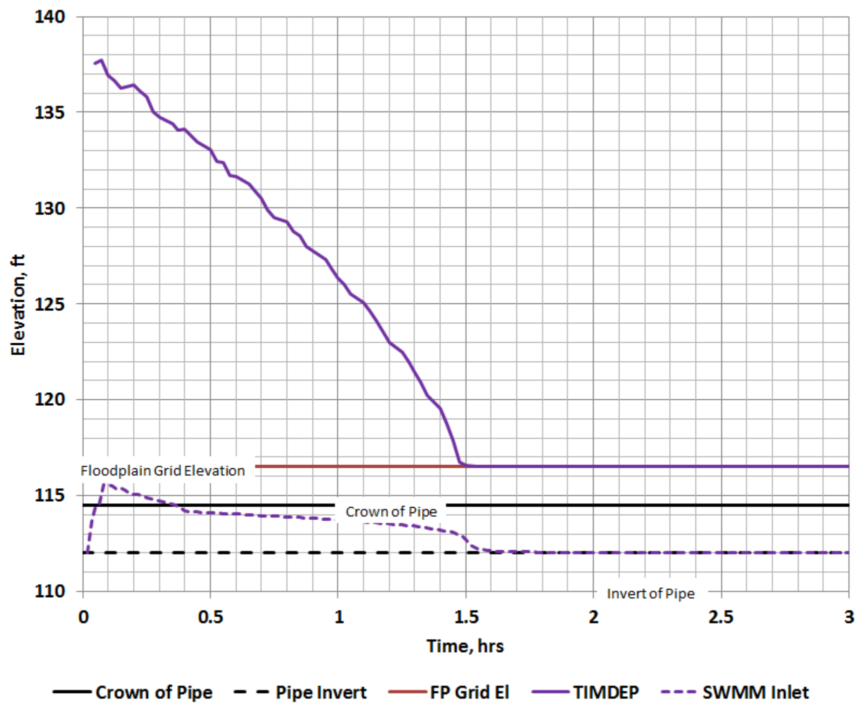
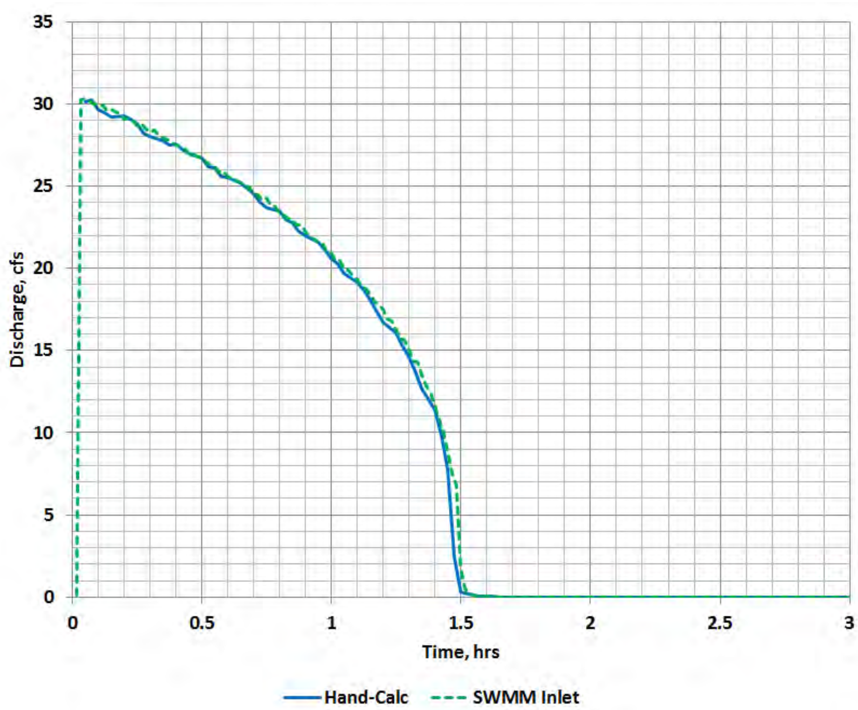


Figure 4.171 Scenario 16: Inlet Type 3, Inlet Pressure



4.2.7.18 Scenario 17: Inlet Type 3, Weir/Orifice Transition, *HGL* below Soffit, with Curb

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 3 Inlet: Grate (Gutter) Inlet with/without Sag that has flow transitioning between weir and orifice flow and also has a curb. The parameters for this scenario are the same as for Section 4.2.7.17 except that the curb height is set at 0.5 feet. A plan view of the test model is shown on Figure 4.147 and a profile view on Figure 4.148. The inflow hydrograph has a uniform discharge designed to force the transition zone where:

If 0.75 $\leq$ H < 1.8 h

If $Q_w \leq Q_o$ then the discharge = Q_w (weir equation controls)

If $Q_w > Q_o$ then the discharge = Q_o (orifice equation controls)

Benchmark

Direct comparison to the smallest value from the solution of either the weir or orifice equations:

Weir Equations:

$$If\ H\ \leq h\ then\!: Q_w = CPH^m = 3.0 * 4.71 * 1.152^{1.5} = 23.5\ cfs$$

$$If\ H > h\ then\!: Q_w = CP\left(H + \frac{h}{2}\right)^m = 3.0 * 4.71 * \left(1.152 + \frac{0}{2}\right)^{1.5} = 23.5 \text{ cfs}$$

Orifice Equations:

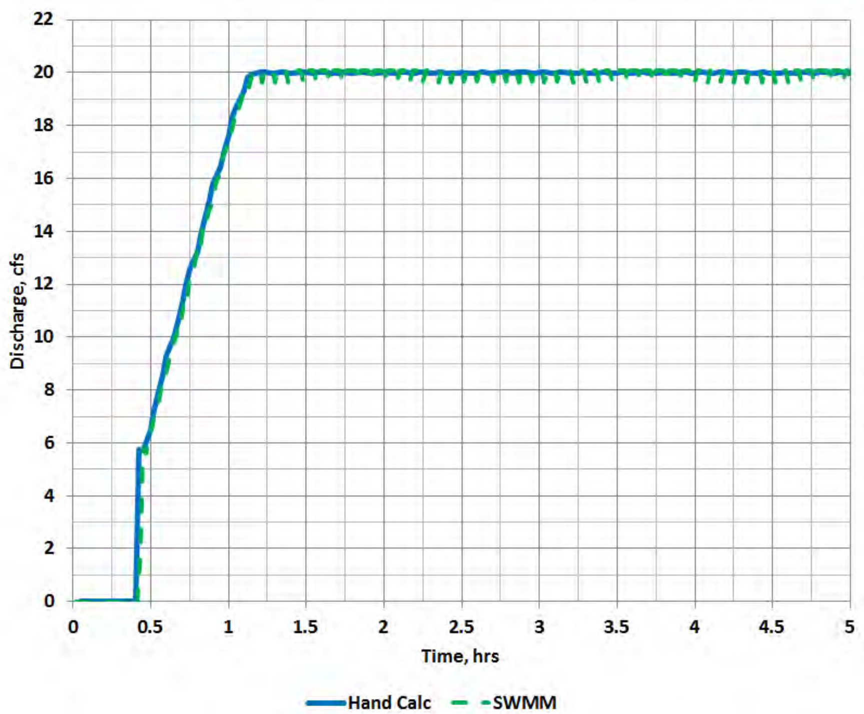
$$If\ H\ \leq h\ then\!: Q_o = C_d\, A\sqrt{2\, g\, H} = 0.67 * 3.14 * \sqrt{2 * 32.2 * 1.4} = 20.0 \text{ cfs}$$

$$If\ H\ > h\ then\!: Q_o = C_d\, A\sqrt{2\, g\left(H + \frac{h}{2}\right)} = 0.67 * 3.14 * \sqrt{2 * 32.2\left(1.4 + \frac{0}{2}\right)} = 20.0 \text{ cfs}$$

Discussion

The flow rate in the storm drain as reported by *SWMM* is equal to 20.0 cfs, which matches the hand calculation of the orifice equation. The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. Comparison for the scenario is shown on Figure 4.172. This approach for modeling a storm drain with a Type 3 inlet under transition conditions with a curb functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.172 Scenario 17: Inlet Type 3, Transition Flow with a Curb



4.2.7.19 Scenario 18: Inlet Type 4, Rating Table, *WSEL* below soffit

<u>Scenario</u>

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 4 Inlet: Defined by a Stage-Discharge Rating Table. *HY-8* was used to develop the following rating curve:

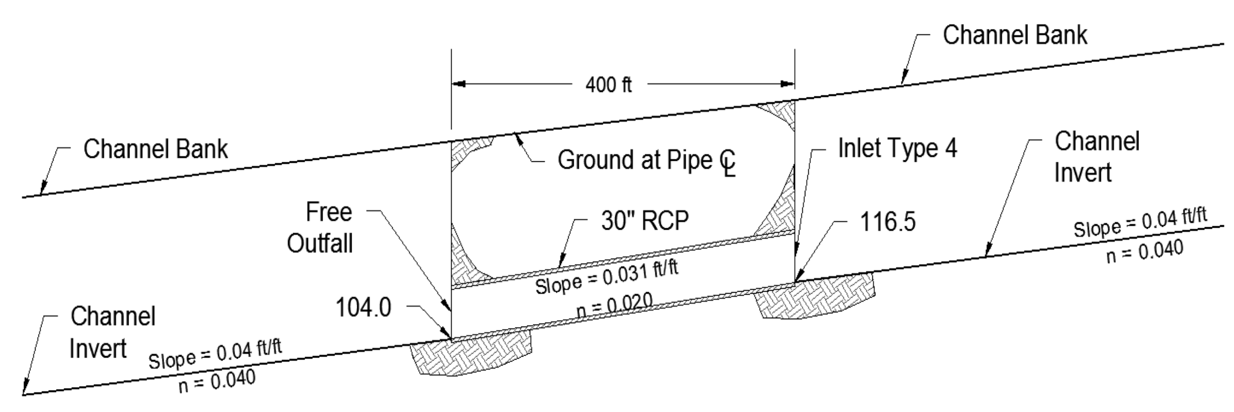
Table 4.42 Type 4 Stage-Discharge Rating

Depth, ft	Discharge, cfs
(1)	(2)
0.0	0.0
0.11	0.20
0.79	3.18
1.12	6.16

Table 4.42 Type 4 Stage-Discharge Rating	
Depth, ft	**Discharge, cfs**
(1)	**(2)**
1.42	9.14
1.69	12.12
1.93	15.0
2.16	18.08
2.39	21.06
2.63	24.04
2.88	27.02
3.15	30.0

Figure 4.164 shows the plan view of the test model and Figure 4.173 shows the profile view.

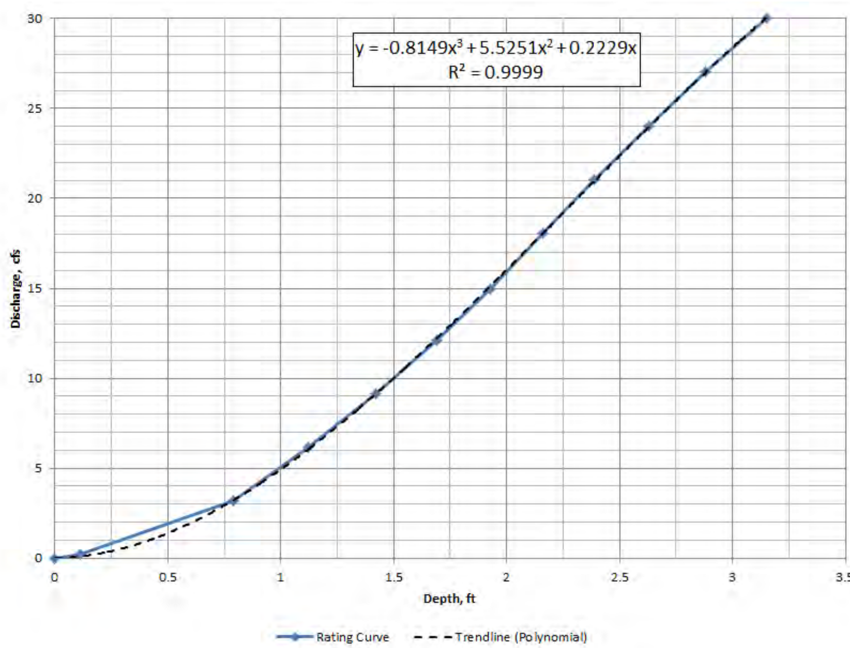
Figure 4.173 Profile View of Inlet Type 4 Test Model



Benchmark

Results were compared to the rating table by using MS Excel to determine the equation of a polynomial line through the stage-discharge rating data (see Figure 4.174). The TIMDEP.OUT file does not include discharge, only depth over time. The depth results from the TIMDEP.OUT file at grid 746 (at the *SWMM* inlet node) were then used to solve the derived equation for discharge and the results plotted on Figure 4.174.

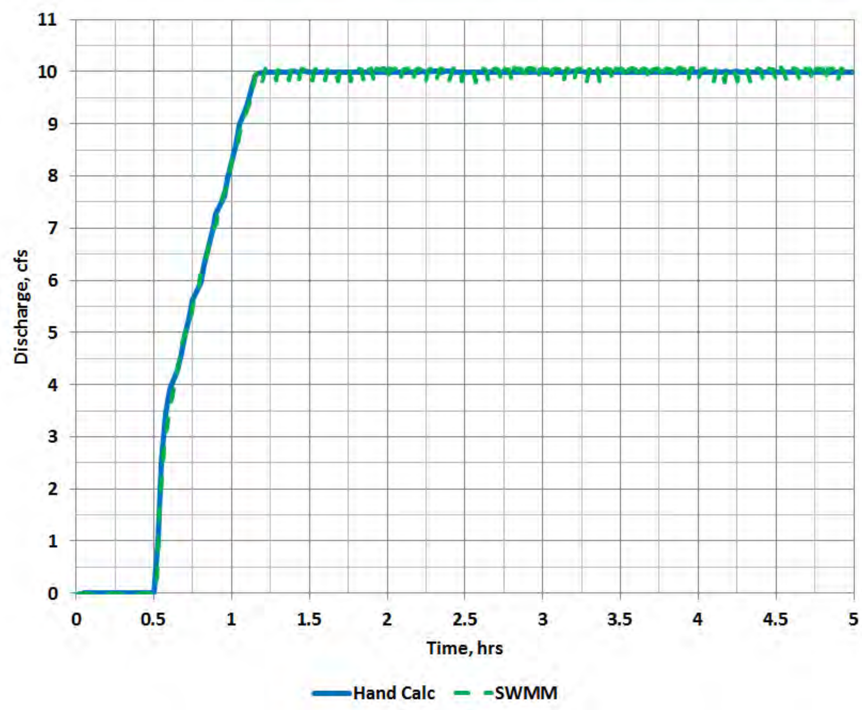
Figure 4.174 Scenario 18: Inlet Type 4 Stage-Discharge Rating Curve



Discussion

The unsteady flow rate in the storm drain as reported by *SWMM* matches the head-discharge relationship for the rating curve (see Figure 4.175). This approach for modeling a storm drain with a Type 4 functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.175 Scenario 18: Inlet Type 4



4.2.7.20 Scenario 19: Inlet Type 4, Rating Table, *WSEL* above soffit, *HGL* below Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 4 Inlet: Defined by a Stage-Discharge Rating Table with ponding on the inlet. The inlet has the same parameters as the previous scenario.

Benchmark

Direct comparison to the rating table.

Discussion

The water surface elevation in the storm drain and on the floodplain grid is shown on Figure 4.176. A comparison to a Type 4 rating curve matched the results of the model and this is shown on Figure 4.177. The head on the inlet is great enough that return flow does not occur. In this scenario, pressure pipe flow is occurring and pipe hydraulics control the flow through the pipe rather than the pipe opening characteristics. This approach for modeling a storm drain with a Type 4 inlet under inlet pressure conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.176 Scenario 19: Inlet Type 4, Water Surface Elevation

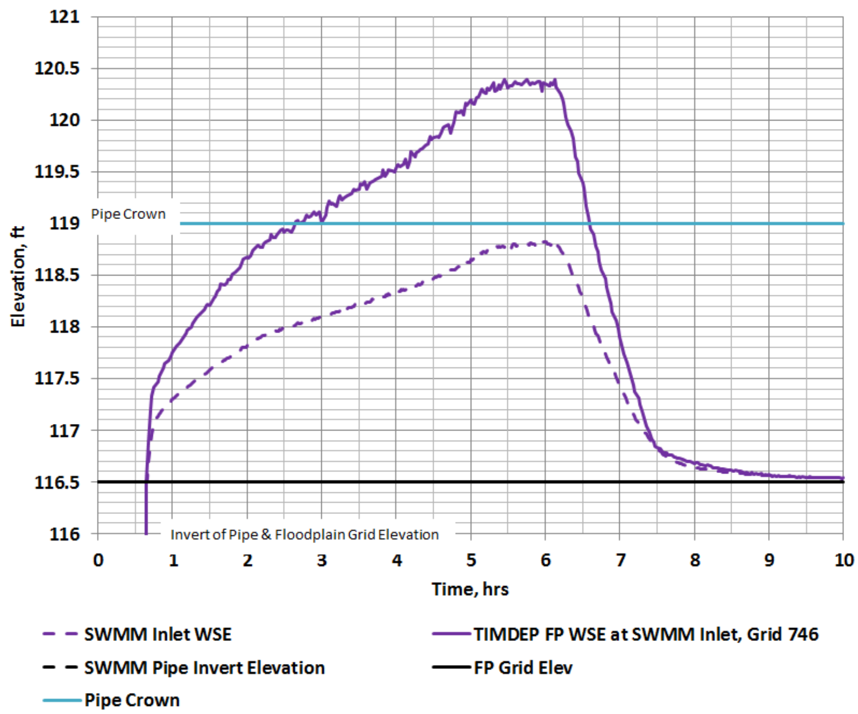
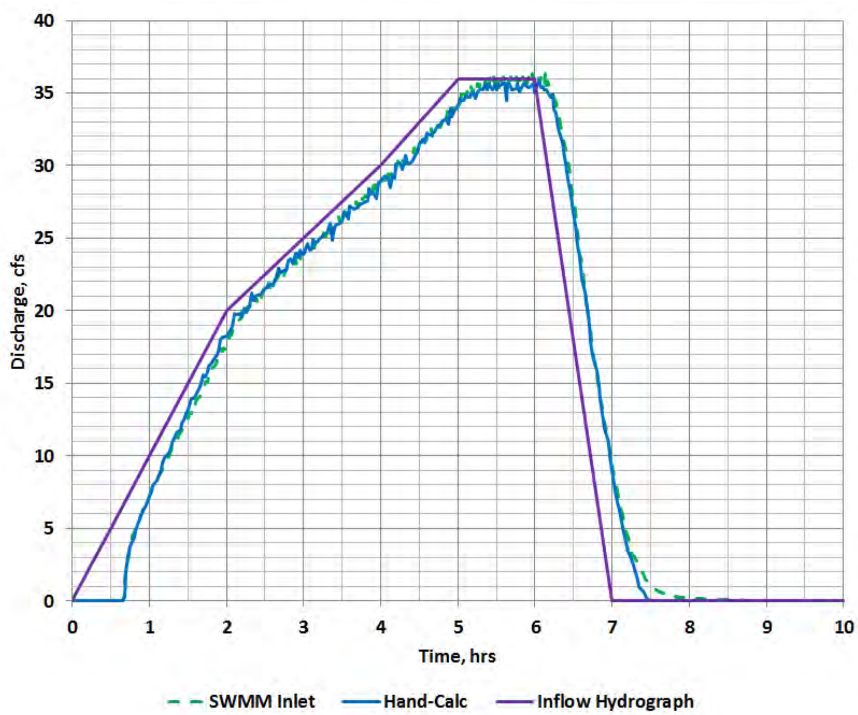


Figure 4.177 Scenario 19: Inlet Type 4, Inlet Pressure



4.2.7.21 Scenario 20: Inlet Type 4, Rating Table, *WSEL* above soffit, *HGL* above Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a Type 4 Inlet: Defined by a Stage-Discharge Rating Table. Inflows into the system are great enough to force pressure flow in the storm drain. The inlet has the same parameters as Section 4.2.7.19.

Benchmark

Direct comparison to the rating table.

Discussion

The water surface elevation in the storm drain and on the floodplain grid is shown on Figure 4.178. A comparison to Type 4 rating table matches the model results and this is shown on Figure 4.179. This approach for modeling a storm drain with a Type 4 inlet under inlet pressure conditions functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.178 Scenario 20: Inlet Type 4, Water Surface Elevation

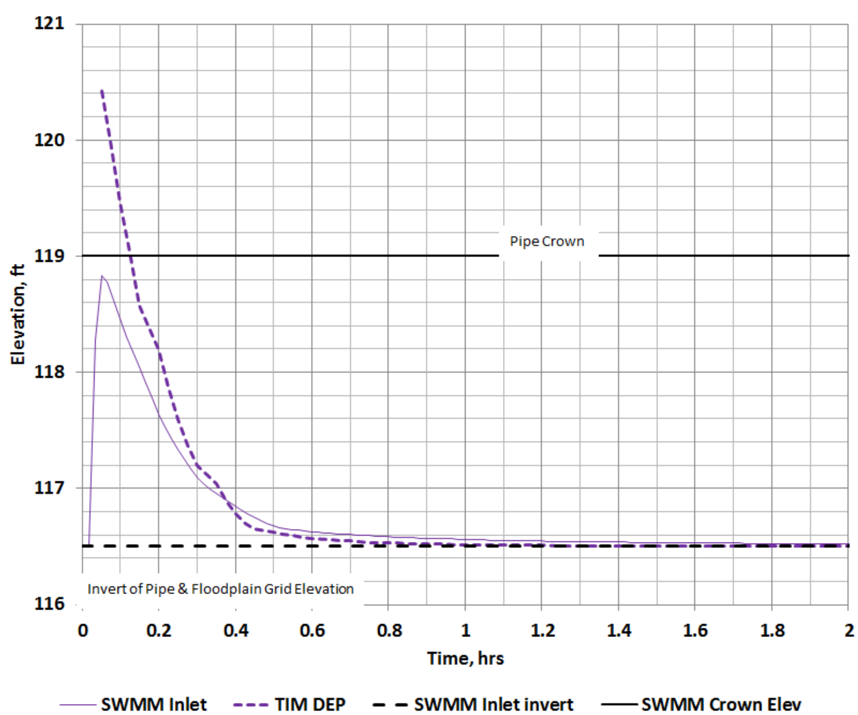
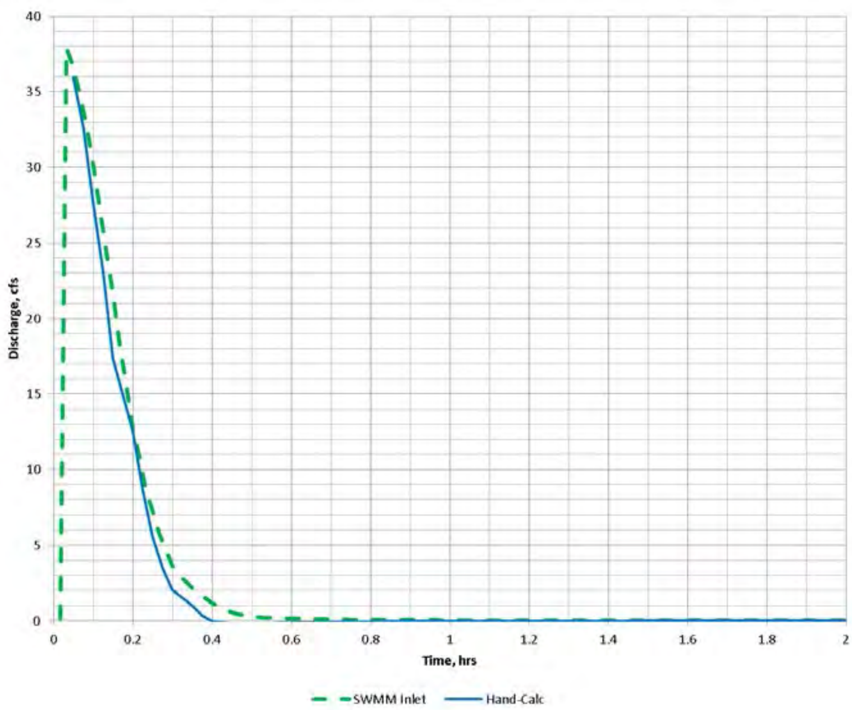


Figure 4.179 Scenario 20: Inlet Type 4, Pressure Pipe



4.2.7.22 Scenario 21: Inlet Type 1, Orifice Flow, *HGL* below Soffit, with Junction

Scenario

Test of *SWMM* system that includes a junction. The inlet type for the storm drain is Type 1: Curb Opening Inlet at Grade and the outlet type is a free outfall to floodplain grids. A plan view of the test model is shown on Figure 4.180 and profile view on Figure 4.181.

Figure 4.180 Plan View of Junction Test Model

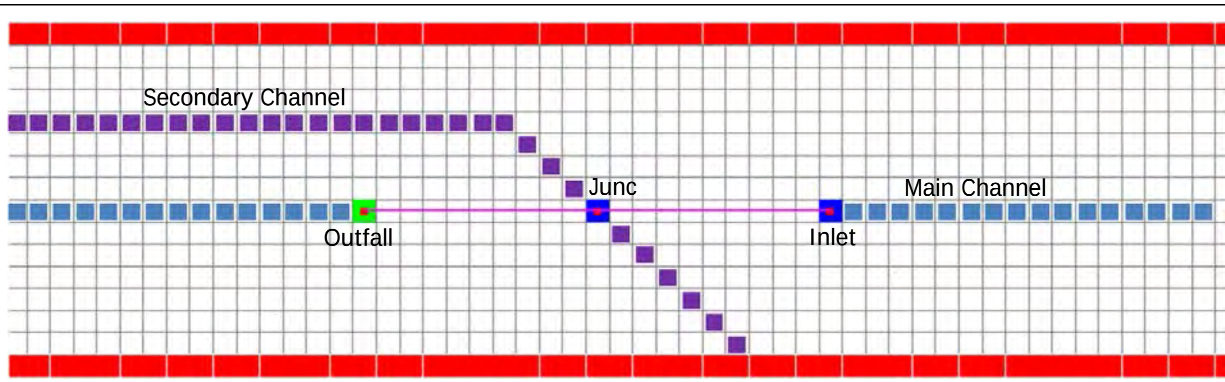
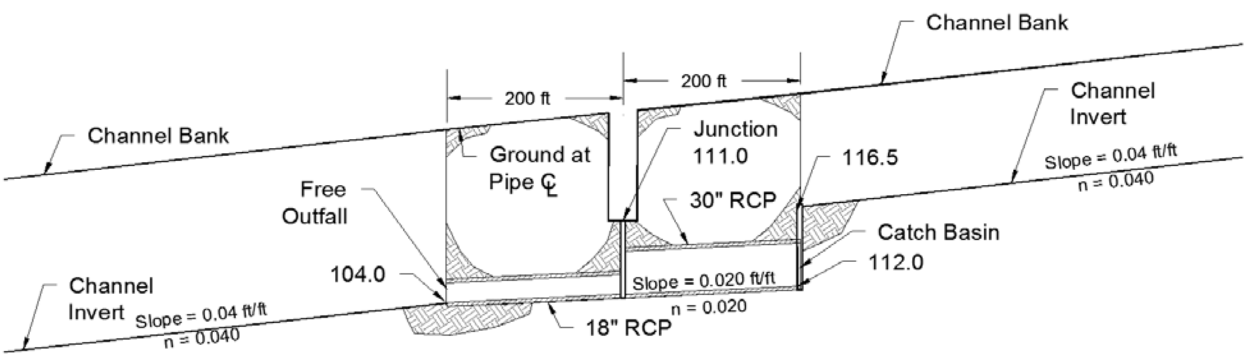


Figure 4.181 Profile View of Junction Test Model



Benchmark

The discharge reported by the *SWMM*.RPT for the *SWMM* inlet inflow, *SWMM* Junction inflow and *SWMM* Outfall Inflow were compared. Also, the *SWMM* Junction flooding was checked to make sure this was zero.

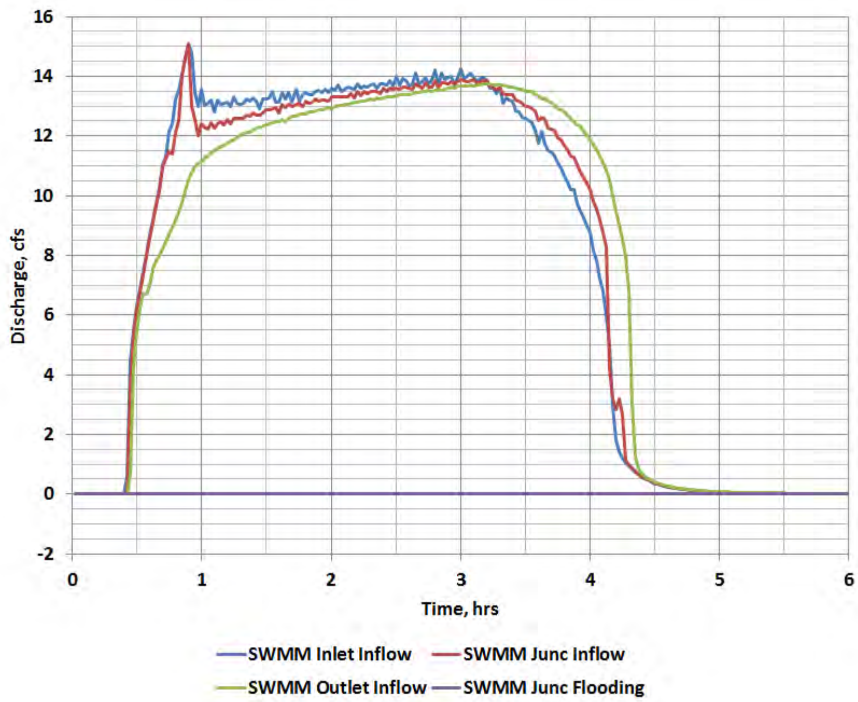
Discussion

As reported by *SWMM*, the inflow for the inlet, junction and outfall reasonably match. Refer to Table 4.43 for a comparison of computed volumes. This approach for modeling a storm drain that includes a junction functions as expected. This is acceptable for *FCDMC* purposes.

Table 4.43 Comparison of volume for Scenario 21

Node (1)	**Volume, 10^6 gal** (2)
SWMM Inlet Inflow	1.262
SWMM Junc Inflow	1.262
SWMM Outlet Inflow	1.263
SWMM Junc Flooding	0

Figure 4.182 Scenario 21: *SWMM* Junction Test Model



4.2.7.23 Scenario 22: Inlet Type 5, Manhole Popped, Low Flow Out of Manhole

Scenario

Test of the *FLO-2D*/*SWMM* approach for *SWMM* manhole popped off and flow out of a manhole opening. The inlet type for the storm drain is Type 1: Curb Opening Inlet at Grade and the manhole is a Type 5: Manhole. The model also includes two channels, a main channel and a secondary channel. The main channel is connected to the *SWMM* inlet and outfall, and the secondary channel passes over the *SWMM* manhole. A plan view of the test model is shown on Figure 4.183 and a profile view on Figure 4.184. The inflow hydrograph has enough discharge to dislodge the manhole cover, but small enough that once the manhole is dislodged only a small amount of water leaves the manhole. The amount of water leaving the manhole is calculated by *SWMM* and *FLO-2D* uses this calculation to place flow on the floodplain grid at the manhole. Flow that leaves the manhole flows down the secondary channel. The inlet and manhole have the following parameters:

1. Curb Opening Length = 3 feet
2. Curb Opening Height = 0.5 feet
3. Curb Weir Coefficient = 3.0

4. Curb Height = 0.0 feet
5. Manhole perimeter = 6.28 feet
6. Manhole flow area = 3.14 ft^2
7. Manhole Weir Coefficient = 3.0
8. Surcharge depth that pops the manhole cover = 0.5 ft

Figure 4.183 Plan View of Inlet Type 5 Test Model

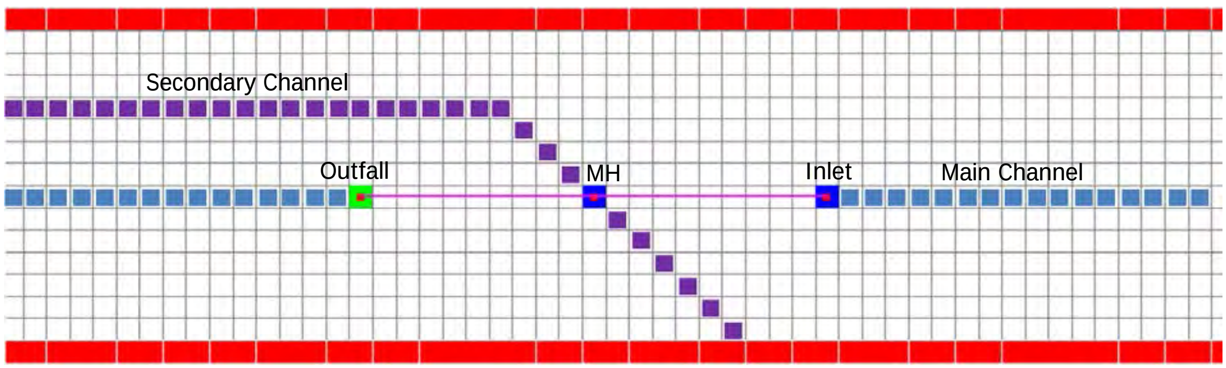
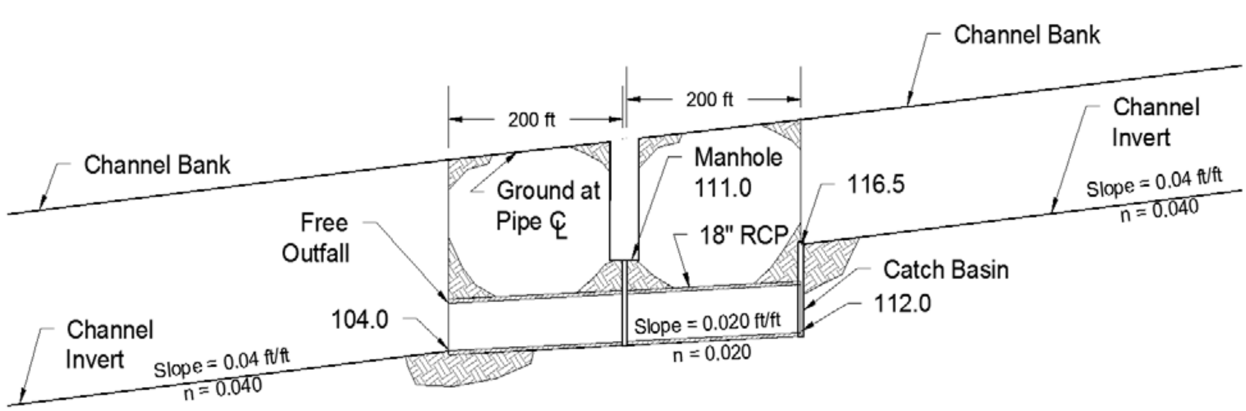


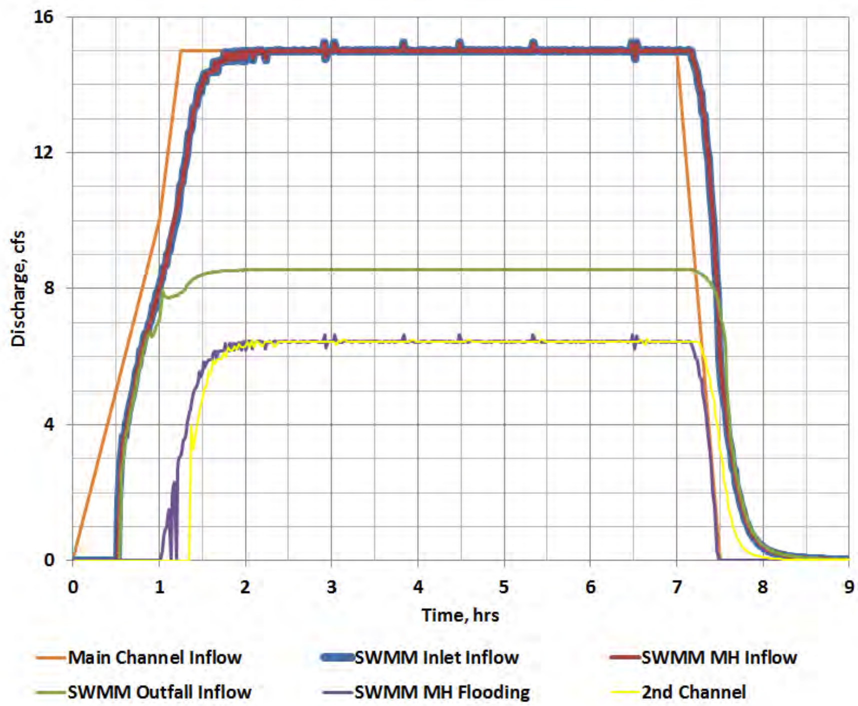
Figure 4.184 Profile View of Inlet Type 5 Test Model



Benchmark

The discharge reported by the *SWMM*.RPT for the *SWMM* inlet and manhole (red and blue lines, respectively, in Figure 4.185) are compared to the inflow hydrograph (orange line). Also, the discharge reported by the *SWMM*.RPT for manhole flooding (purple line) is compared to the discharge reported in HYCROSS.OUT for a grid in the secondary channel (yellow line).

Figure 4.185 Scenario 22: Inlet Type 5, Low Flow Out of a Manhole Opening



Discussion

As reported by *SWMM*, the flow entering the inlet and manhole reasonably match the inflow hydrograph. Also, the flow leaving the manhole through the cover reasonably matches the flow in the secondary channel. This approach for modeling a storm drain with water exiting a manhole functions as expected. This is acceptable for *FCDMC* purposes.

4.2.7.24 Scenario 23: Inlet Type 5, Manhole Popped, High Flow Out of Manhole

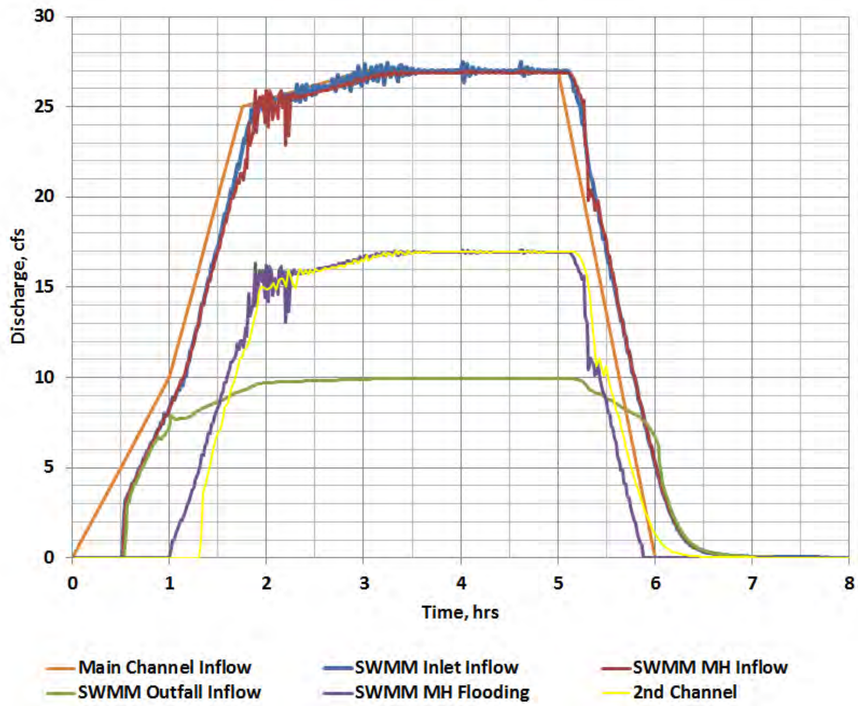
Scenario

Test of the *FLO-2D*/*SWMM* approach for *SWMM* manhole popped off and a large amount of flow out of a manhole opening. This model has the same parameters as Section 4.2.7.23, except for the inflow hydrograph. The inflow hydrograph is shown on Figure 4.186.

Benchmark

The discharge reported by the *SWMM*.RPT for the *SWMM* inlet and manhole (red and blue lines, respectively) are compared to the inflow hydrograph (orange line). Also, the discharge reported by the *SWMM*.RPT for manhole flooding (purple line) is compared to the discharge reported in HYCROSS.OUT for a grid in the secondary channel (yellow line). Oscillations are due to the difference in computation time step between FLO-2D and SWMM.

Figure 4.186 Scenario 23: Inlet Type 5, High Flow Out of a Manhole Opening



<u>Discussion</u>

As reported by *SWMM*, the flow entering the inlet and manhole reasonably matches the inflow hydrograph. Also, the flow leaving the manhole through the cover reasonably matches the flow in the secondary channel. This approach for modeling a storm drain with water exiting a manhole functions as expected. This is acceptable for *FCDMC* purposes.

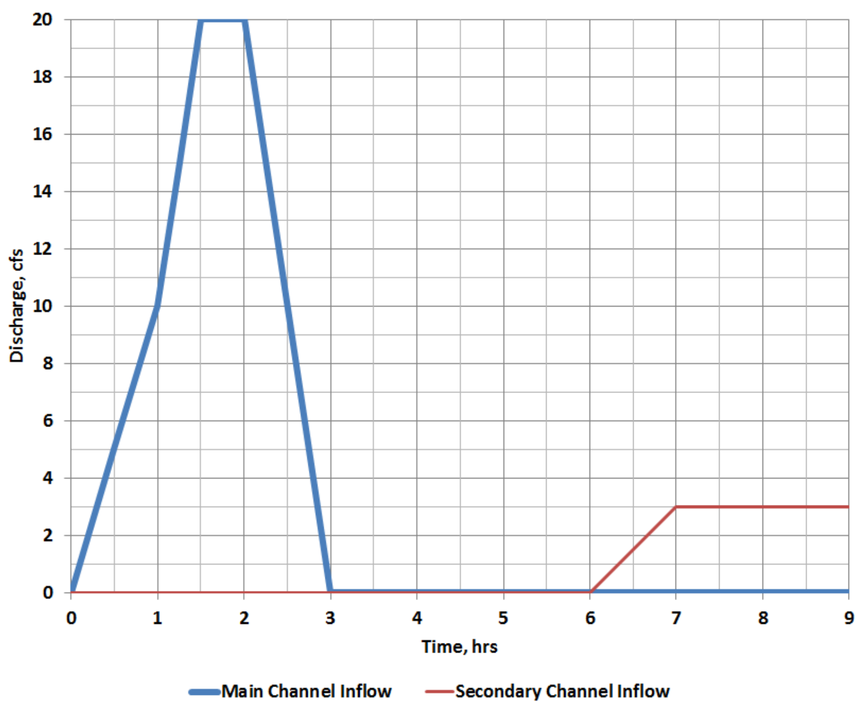
4.2.7.25 Scenario 24: Inlet Type 5, Manhole Popped, Weir Flow into Manhole

<u>Scenario</u>

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids and with a manhole located between the floodplain grids. The inlet type for the storm drain is Type 1 and the manhole is a Type 5 Inlet: Curb Opening Inlet at Grade and the manhole is a Type 5 Inlet. A plan view of the test model is shown on Figure 4.183 and a profile view on Figure 4.184 shows the profile view. The inflow hydrograph to the *SWMM* storm drain has enough discharge to dislodge the manhole cover. Once the cover is popped off, flow in the main channel is stopped and flow in the secondary channel starts.

Flow in the secondary channel is small enough that only weir flow occurs for flow into the manhole opening. This occurs when the depth on the floodplain grid is less than 0.75 ft. The inflow hydrographs are shown on Figure 4.187.

Figure 4.187 Scenario 24: Inflow Hydrographs



Benchmark

Direct comparison to the smaller of the two weir equations for a Type 3 Inlet.

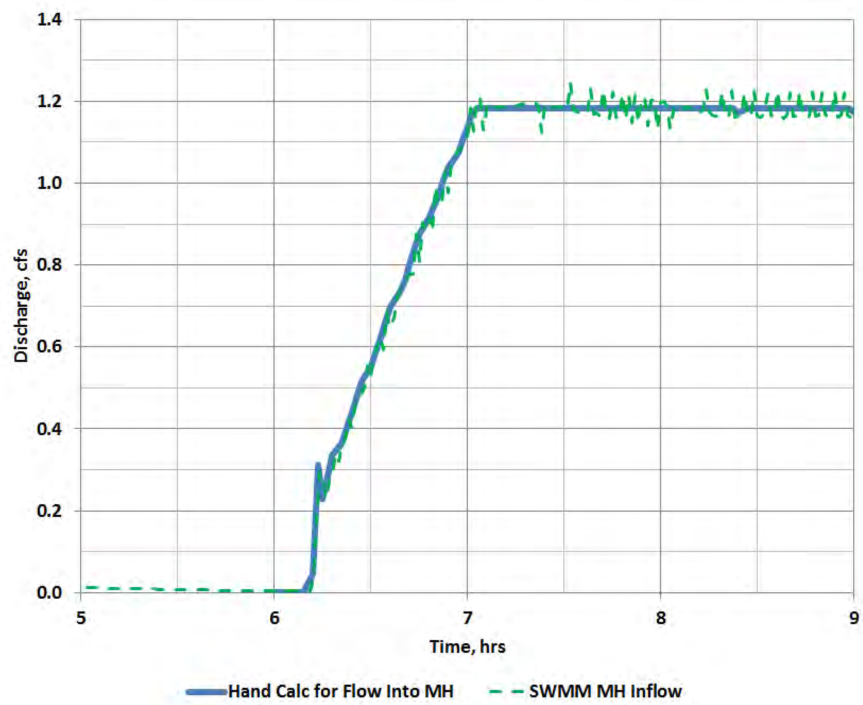
$$If\ H\ \leq h\ then: Q_w = CPH^m = 3.0 * 6.28 * 0.158^{1.5} = 1.18\ cfs$$

$$If\ H > h\ then: Q_w = CP\left(H + \frac{h}{2}\right)^m$$

Discussion

The hand calculation for flow into the manhole opening matches the results report by *SWMM*.RPT for flow into the manhole (see Figure 4.188). This approach for modeling a storm drain with water entering a manhole functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.188 Scenario 24: Inlet Type 5, Weir Flow into a Manhole Opening



4.2.7.26 Scenario 25: Inlet Type 5, Manhole Popped, Orifice Flow into Manhole

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids and with a manhole located between the floodplain grids. The inlet type for the storm drain is Type 1: Curb Opening Inlet at Grade and the manhole is a Type 5 Inlet. A plan view of the test model is shown on Figure 4.183 and a profile view on Figure 4.189. The inflow hydrograph to the *SWMM* storm drain has enough discharge to dislodge the manhole cover. Once the cover is popped off, flow in the main channel is stopped and flow in the secondary channel starts. Flow in the secondary channel is large enough that orifice flow occurs for flow into the manhole. This occurs when the depth on the floodplain grid is greater than 1.8 ft. The inflow hydrographs are shown on Figure 4.190.

Manhole opening height (h) = 2.0 ft

Area of manhole opening (A) = 3.1416 sf

FLO-2D flow depth on grid element (H) after hour 7 = 1.09 ft

Figure 4.189 Profile View of Inlet Type 5 Orifice Flow Test Model

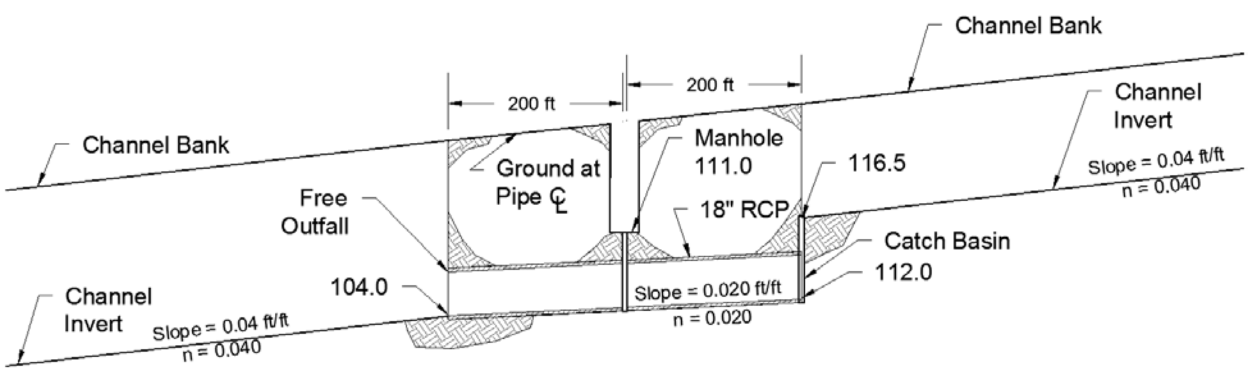
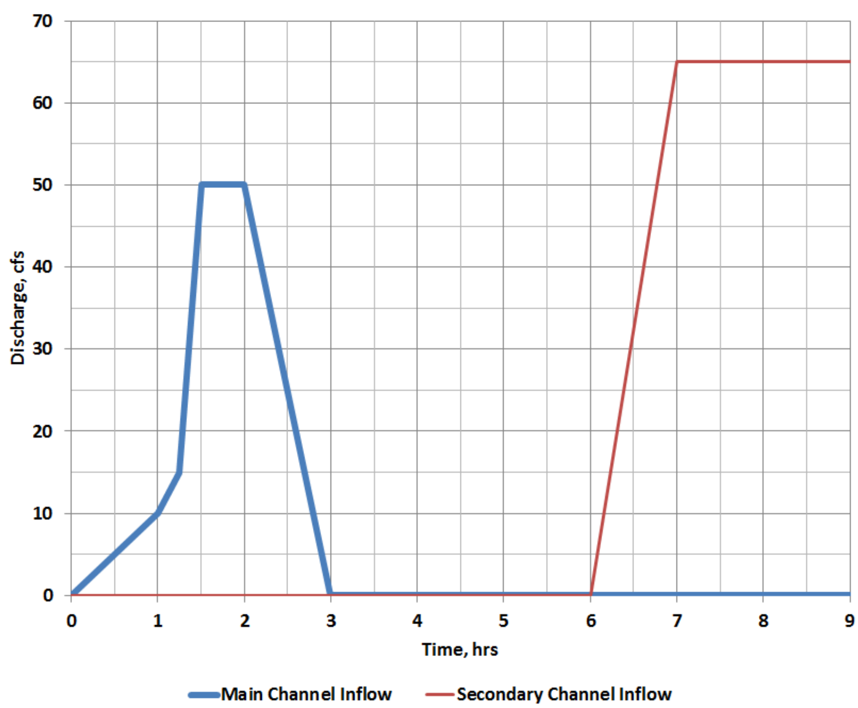


Figure 4.190 Scenario 25: Inflow Hydrographs



Benchmark

Direct comparison to the smaller of the Orifice Equations for a Type 3 Inlet (17.64 cfs controls):

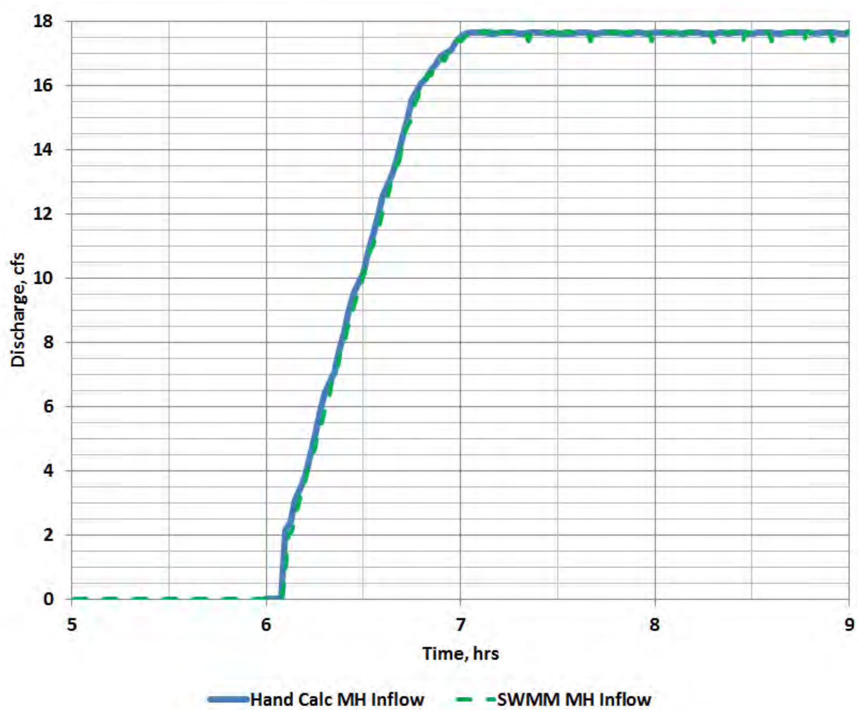
$$If\ H\ \leq h\ then{:}\ Q_o = \ C_d\ A\sqrt{2\ g\ H} = 0.67 * 3.1416 * \ \sqrt{2 * 32.2 * 1.09} = \ 17.64\ \text{cfs}$$

$$If\ H\ > h\ then{:}\ Q_o = \ C_d\ A\sqrt{2\ g\ \left(H + \frac{h}{2}\right)} = 0.67 * 3.1416\sqrt{2 * 32.2\ \left(1.09 + \frac{2}{2}\right)} = \ 24.42\ \text{cfs}$$

Discussion

The hand calculation by time interval for flow into the manhole opening matches the results reported by the *SWMM*.RPT for flow into the manhole (see Figure 4.191). The hand calculation check was also done for the rising limb of the hydrograph before the uniform flow rate was reached. The *SWMM* results also match those hand calculations. This approach for modeling a storm drain with water entering a manhole functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.191 Scenario 25: Inlet Type 5, Orifice Flow into a Manhole Opening

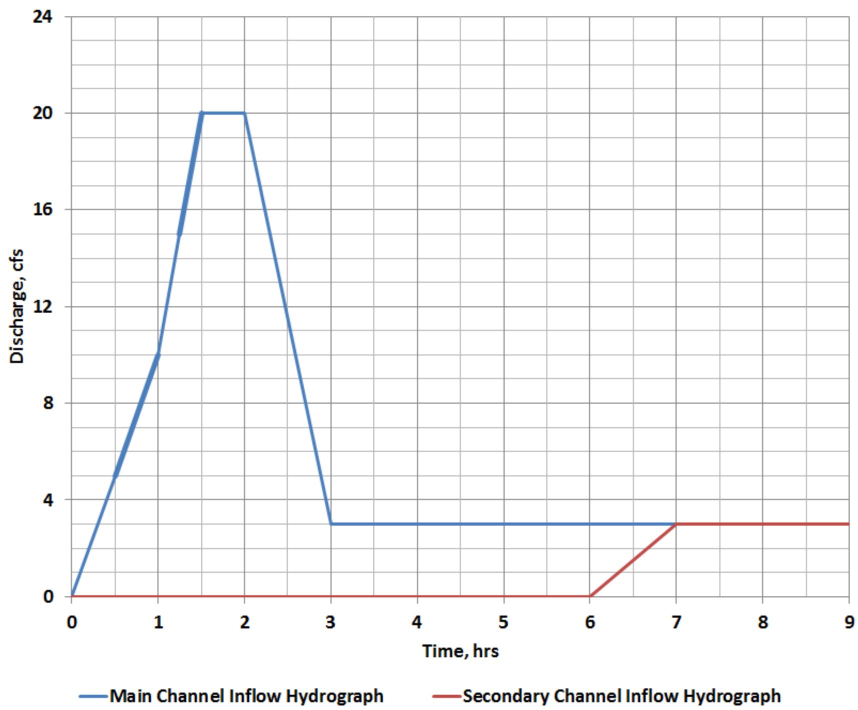


4.2.7.27 Scenario 26: Inlet Type 5, Manhole Popped, Floodplain Grid *HGL* > Pipe *HGL*

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids and with a manhole located between the floodplain grids. The inlet type for the storm drain is Type 1: Curb Opening Inlet at Grade and the manhole is a Type 5 Inlet. A plan view of the test model is shown on Figure 4.183 and a profile view on Figure 4.184. The inflow hydrograph to the *SWMM* storm drain has enough discharge to dislodge the manhole cover. Once the cover is popped off, flow in the main channel is set to 3 cfs and flow in the secondary channel is set to 3 cfs. Flow in the secondary channel is small enough that weir flow occurs for flow into the manhole. The inflow hydrographs are shown on Figure 4.192.

Figure 4.192 Scenario 26: Inflow Hydrographs



Benchmark

Direct comparison to the smaller of the two weir equations plus the inlet storm drain flow for a Type 3 Inlet.

$$If\ H\ \leq h\ then: Q_w = CPH^m = 3.0 * 6.28 * 0.165^{1.5} = 1.26\ cfs$$

$$If\ H > h\ then: Q_w = CP\left(H + \frac{h}{2}\right)^m$$

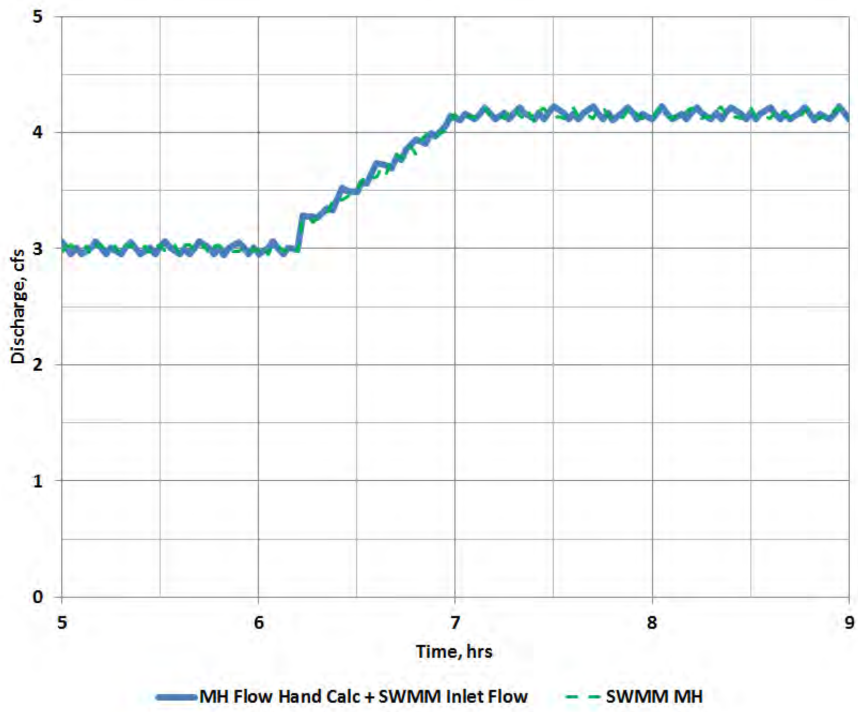
Inlet storm drain flow = 3 cfs

Total storm drain flow at the manhole = 1.26 + 3 = 4.26 cfs

Discussion

The hand calculation for flow into the manhole opening matches the results report by *SWMM*.RPT for flow into the manhole. This approach for modeling a storm drain with water entering a manhole functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.193 Scenario 26: Inlet Type 5, Floodplain Grid *HGL* > Pipe *HGL*

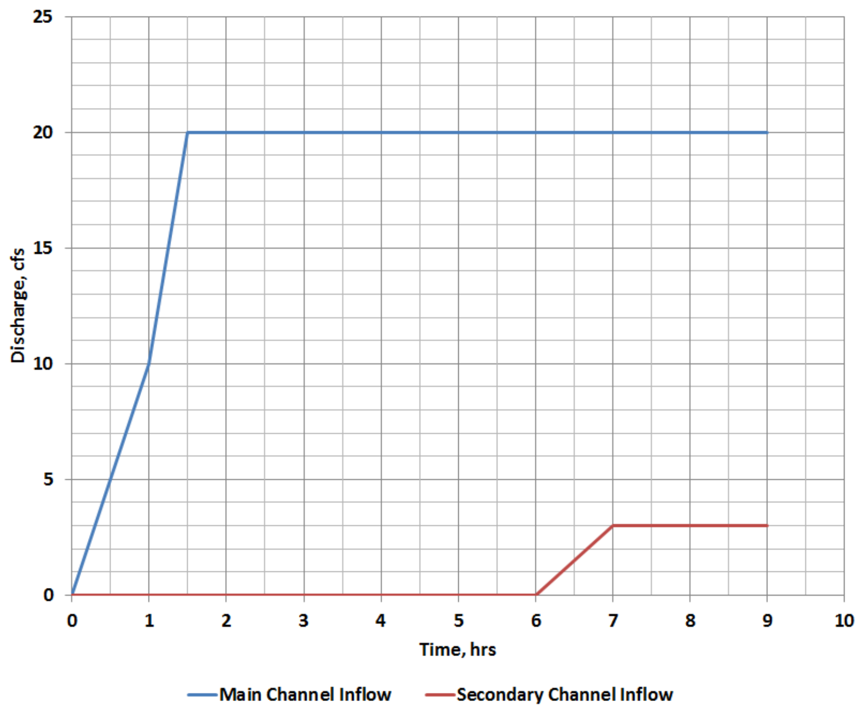


4.2.7.28 Scenario 27: Inlet Type 5, Manhole Popped, Floodplain Grid *HGL* < Pipe *HGL*

Scenario

Test of the *FLO-2D/SWMM* approach for a storm drain connecting two floodplain grids and with a manhole located between the floodplain grids. The inlet type for the storm drain is Type 1: Curb Opening Inlet at Grade and the manhole is a Type 5 Inlet. A plan view of the test model is shown on Figure 4.183 and a profile view on Figure 4.184. The inflow hydrograph to the *SWMM* storm drain has enough discharge (20 cfs) to dislodge the manhole cover. Once the cover is popped off flow in the main channel does not change and flow in the secondary channel is set to 3 cfs. Flow in the secondary channel is small enough that it does not prevent flow coming out of the manhole. The total flow downstream from the manhole in the secondary channel is the sum of the secondary channel inflow and the flow forced out of the manhole. A Floodplain Grid Courant Number of 0.2 is used. The inflow hydrographs are shown on Figure 4.194.

Figure 4.194 Scenario 27: Inflow Hydrographs



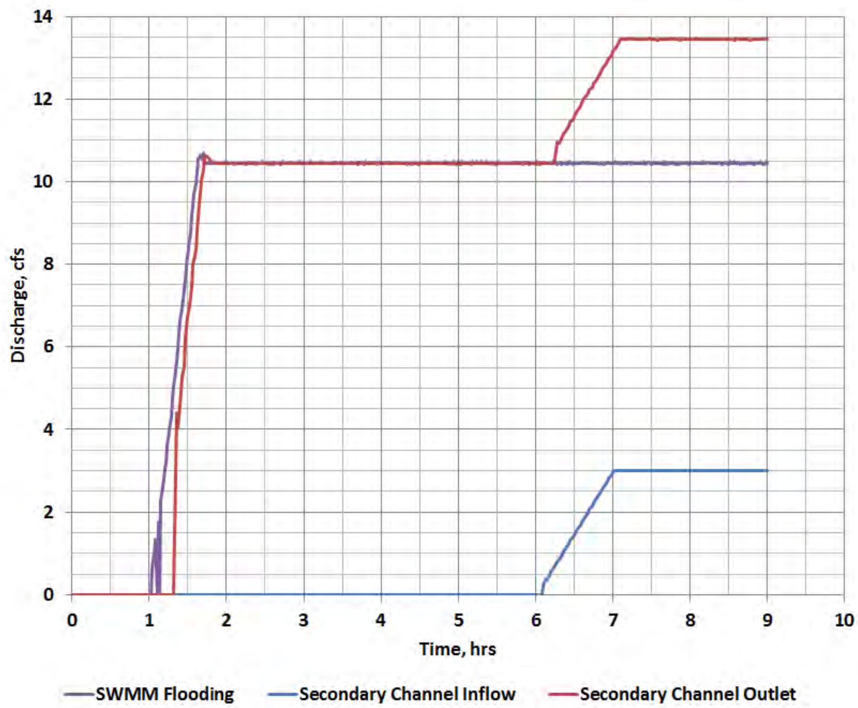
Benchmark

The sum of the manhole flooding report in the *SWMM*.RPT file (purple line) plus the secondary channel flows upstream of the manhole (blue line, see Figure 4.195) are compared to the flows in the secondary channel downstream of the manhole (red line).

Discussion

The summation of the manhole flooding and the secondary channel inflow matches the flow in the secondary channel downstream of the manhole. This approach for modeling a storm drain with water exiting a manhole functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.195 Scenario 27: Inlet Type 5, Floodplain Grid *HGL* < Pipe *HGL*

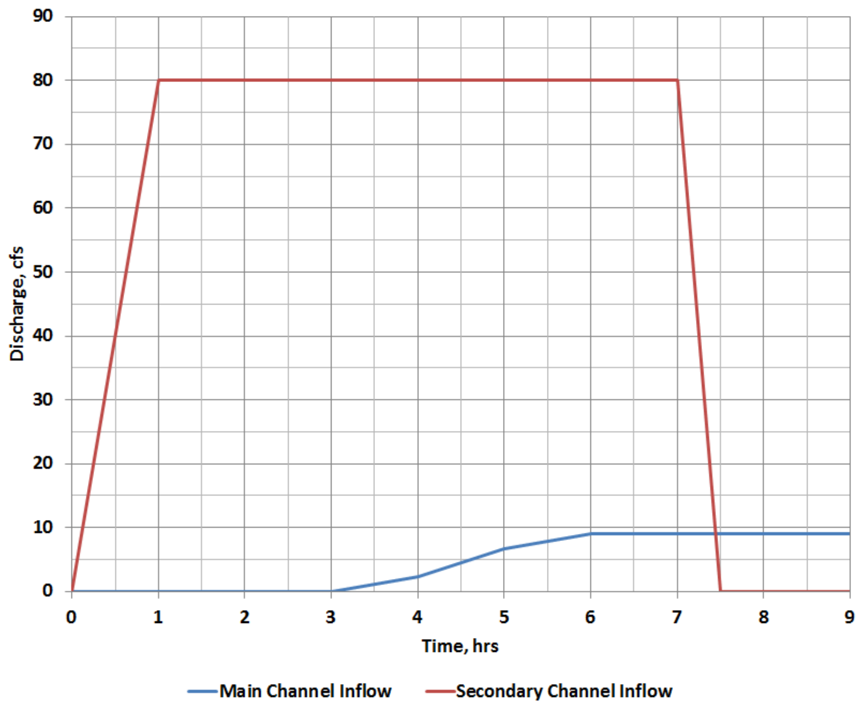


4.2.7.29 Scenario 28: Inlet Type 5, Flow on Floodplain Does Not Allow Manhole to Pop

Scenario

Test of the *FLO-2D/SWMM* approach for a storm drain connecting two floodplain grids and with a manhole located between the floodplain grids. The inlet type for the storm drain is Type 1: Curb Opening Inlet at Grade and the manhole is a Type 5 Inlet. A plan view of the test model is shown on Figure 4.183 and a profile view on Figure 4.184. The inflow hydrograph to the secondary channel has enough pressure to prevent the manhole cover from dislodging. Then the flow in the secondary channel is set to zero and the manhole cover is allowed to dislodge. The inflow hydrographs are shown on Figure 4.196.

Figure 4.196 Scenario 28: Inflow Hydrographs



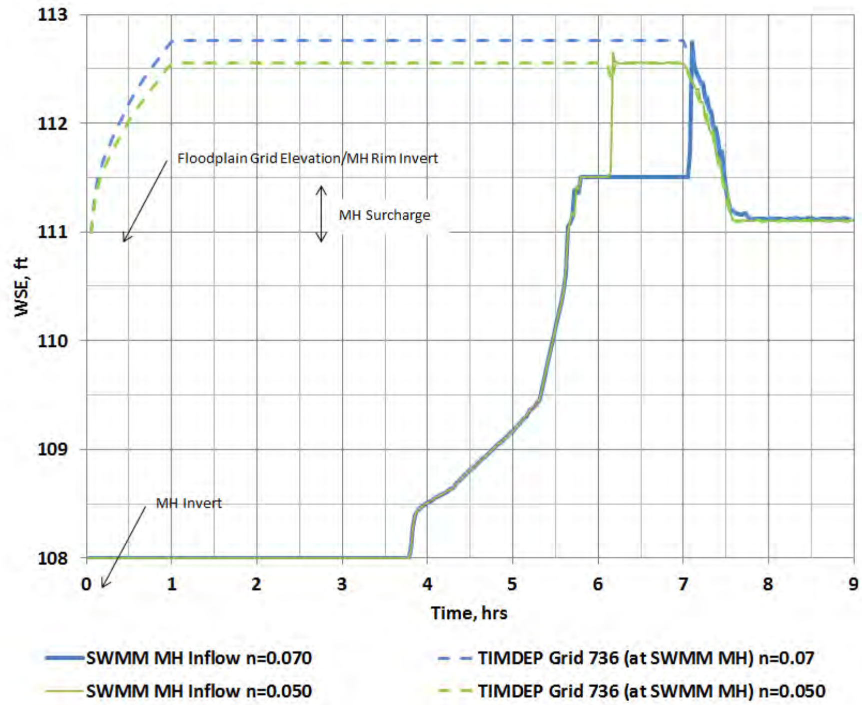
Benchmark

The file STORMDRAIN.CHK is checked to see when the manhole is popped.

Discussion

Two models were developed with only one difference, the *n*-value at the manhole (0.05 to 0.07). The increase in the roughness was used to slightly increase the *WSEL* at the manhole. The results are shown on Figure 4.196. For the model with an *n*-value of 0.05 and a *WSEL* of 112.56 ft the manhole popped at 6.15 hrs, just prior to stopping the flow in the secondary channel. The early pop of the manhole cover is due to slight instabilities in the *SWMM* component of the model primarily caused by the difference in computation time step between *SWMM* and *FLO-2D*. For the model with a *n*-value of 0.07 and a *WSEL* of 112.76 ft the manhole popped at 7.05 hrs, just after stopping the flow in the secondary channel. This approach for modeling a storm drain flow over the manhole preventing the cover from popping off functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.197 Scenario 28: Inlet Type 5, Flow on Floodplain Does Not Allow Manhole to Pop

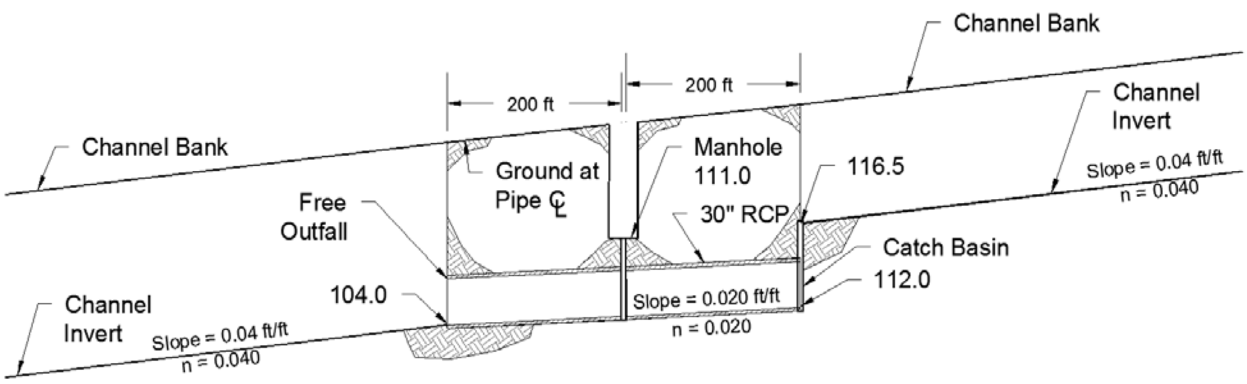


4.2.7.30 Scenario 29: Inlet Type 5, Manhole Popped, Surcharged Pipe, Bypass Flow

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain connecting two floodplain grids with a manhole. The manhole is popped and then flow is added to the secondary channel that is large enough that some flow continues past the manhole. The purpose of this test is to check the amount of flow entering the storm drain through the manhole opening plus flow in the secondary channel downstream of the manhole is equivalent to the total flow entering the secondary channel. Also, the amount of flow entering the manhole is great enough that the storm drain is under surcharged conditions. A plan view of the test model is shown on Figure 4.183 and a profile view on Figure 4.198.

Figure 4.198 Scenario 29: Profile View



Benchmark

The check for this scenario is the sum of the flow entering the storm drain plus the flow continuing past the manhole equals the total flow entering the secondary channel.

Discussion

The flow entering the storm drain was obtained from *SWMM*.RPT file for the manhole node. The flow continuing past the manhole was obtained from the HYCROSS file. These are shown on Figure 4.199 along with the summation and the total flow entering the secondary channel. The surcharge on the storm drain is shown on Figure 4.200. This approach for modeling a storm drain with a manhole under inlet pressure conditions with flow past the manhole functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.199 Scenario 29: Type 1 with Bypass Flow

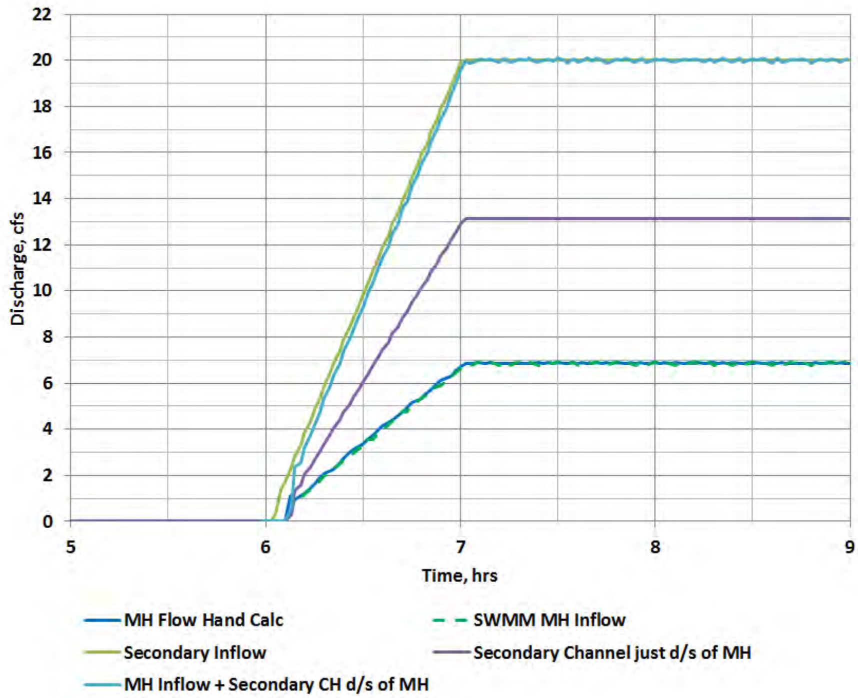
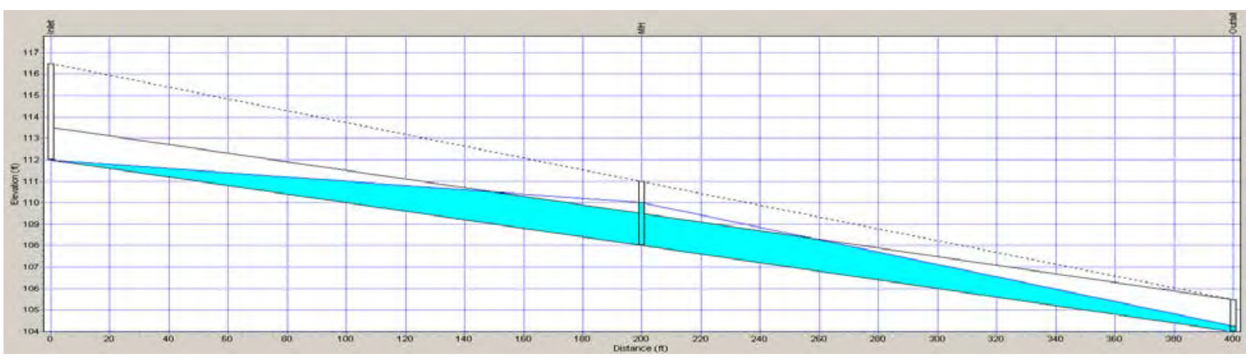


Figure 4.200 Scenario 29: Storm Drain Profile



All models below this point have the following parameters for the Type 1 inlet:

1. Curb Opening Length = 3 feet
2. Curb Opening Height = 0.5 feet
3. Weir Coefficient = 3.0
4. Curb Height = 0.0 feet

Any other parameter that was changed for each specific model with be mentioned in the <u>Scenario</u> section of each model.

4.2.7.31 Scenario 30: Outfall to Lake Submerged Floodplain

<u>Scenario</u>

Test of the *FLO-2D*/*SWMM* approach for a storm drain outfall submerged in a lake. A Type 1 Inlet: Curb Opening Inlet at Grade that has weir flow was used to connect two grid elements. The main setup of the test model consist of (1) a major wash that is 100 feet wide by 10 feet deep, (2) a parallel minor wash 20 feet wide by 8.5 feet deep and (3) the outfall invert is below the *FLO-2D* ground surface elevation to simulate the submergence on the outfall. A floodplain Courant value of 0.20 was used to stabilize the model because of numeric instability due to the difference in computation time step between *FLO-2D* and *SWMM*. A plan view of the test model is shown Figure 4.201 and Figure 4.202 and a profile view on Figure 4.203. The test was designed to show that water entering the inlet, will be able to outflow back onto the surface from a submerged outfall. An inflow hydrograph was applied only to the minor wash in the model.

Figure 4.201 Scenario 30: Plan View of the Test Model - Elevation

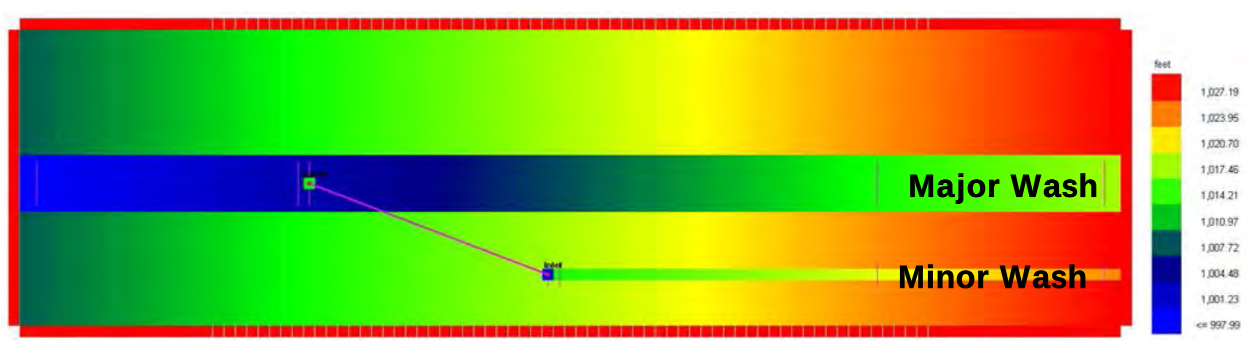


Figure 4.202 Scenario 30: Plan View of the Test Model – Max Depth

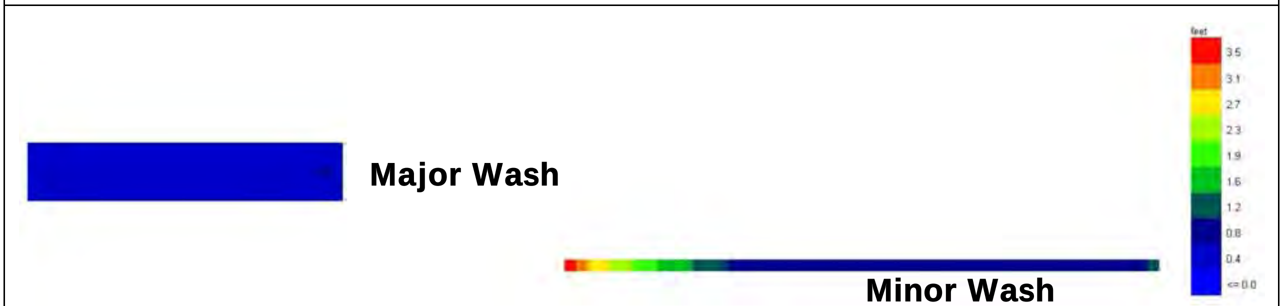
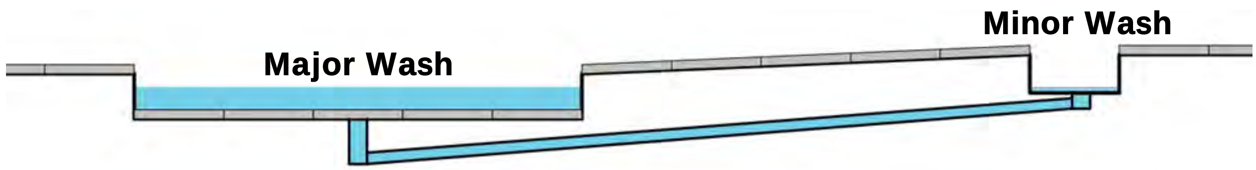


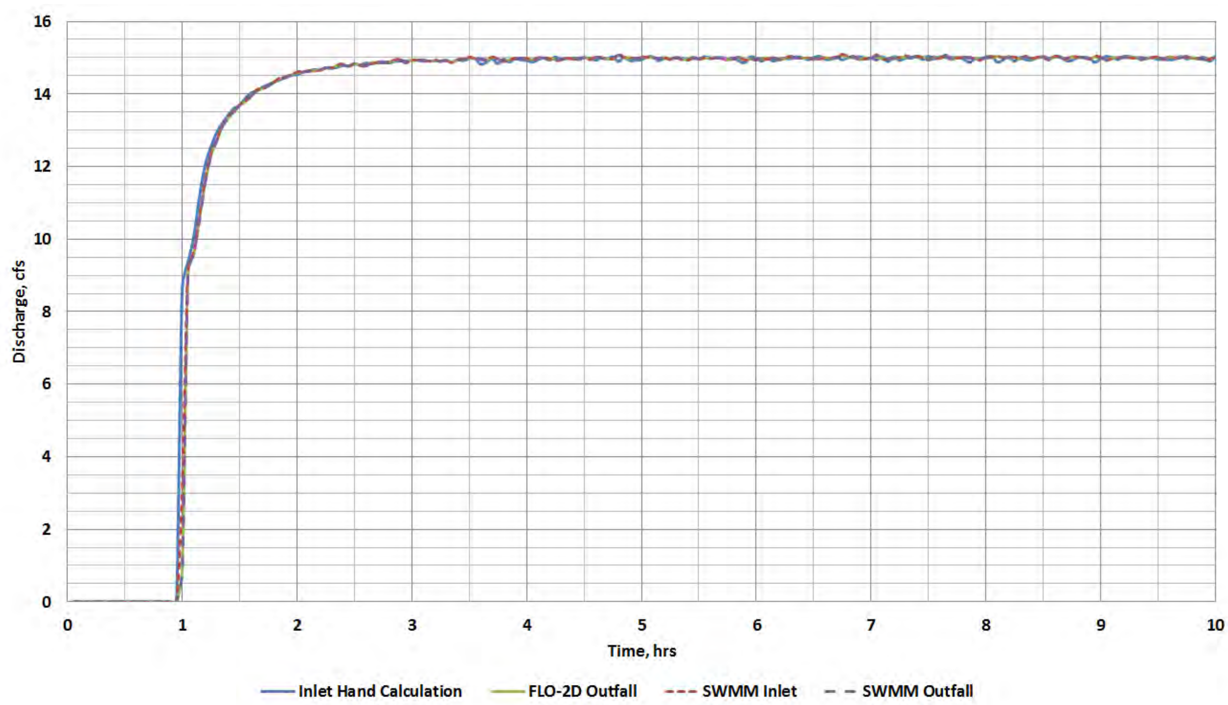
Figure 4.203 Scenario 30: Profile View of the Test Model



Benchmark

The discharge coming into the inlet (blue & red line) should equal the discharge coming out of the outfall (green and purple line). The solid blue line shows the hand calculations that use weir and orifice equations for the Type 1 inlet. This comparison is shown on Figure 4.204.

Figure 4.204 Scenario 30: Inflow and Outfall Hydrographs



Discussion

The discharge in the inlet equals the discharge at the outfall. This shows that an outfall set below the ground surface elevation (submerged) will discharge to the surface under the correct head conditions. This is acceptable for *FCDMC* purposes.

4.2.7.32 Scenario 31: Outfall to Lake Submerged *1D*-Channel

Scenario

This test has the same setup as Scenario 30 above; the only difference is that it uses the *1D*-channel component. A plan view of the test model is shown on Figure 4.205 and Figure 4.206 and a profile view on Figure 4.207. A floodplain Courant value of 0.20 and a *1D*-channel Courant value of 0.60 was used to stabilize the model. The test was designed to show that water entering the inlet, will be able to outflow back onto the surface from a submerged outfall. An inflow hydrograph was applied only to the minor wash in the model.

Figure 4.205 Scenario 31: Plan View of the Test Model - Elevation

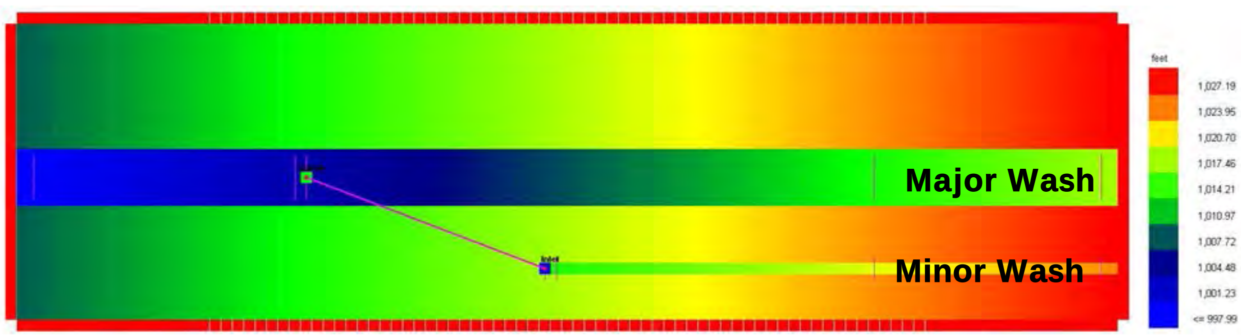
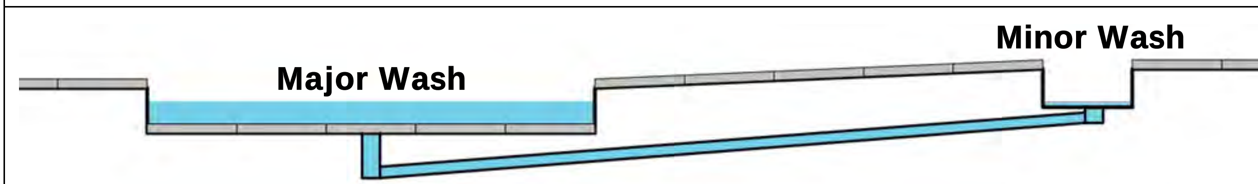


Figure 4.206 Scenario 31: Plan View of the Test Model – Max Depth

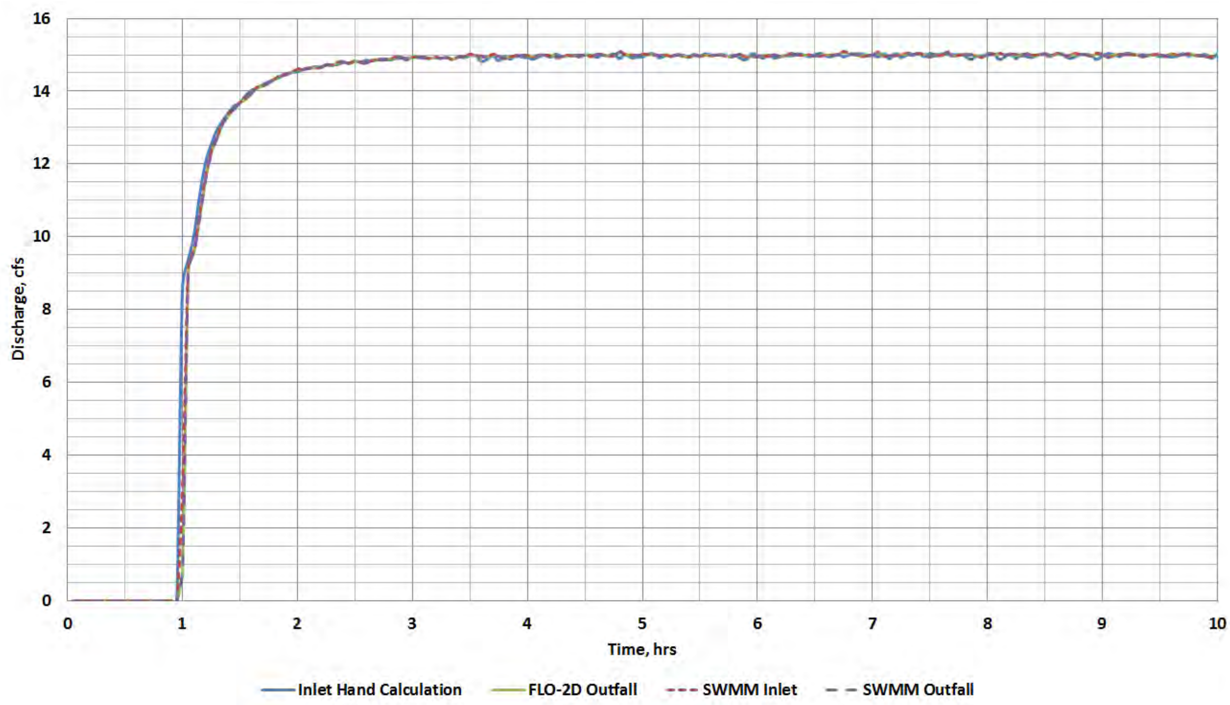
Figure 4.207 Scenario 31: Profile View of the Test Model



Benchmark

The discharge at the inlet (blue & red line) should equal the discharge at the outfall (green and purple line). The solid blue line shows the hand calculations that use the weir and orifice equations for the Type 1 inlet. This comparison is shown on Figure 4.208.

Figure 4.208 Scenario 31: Inflow and Outfall Hydrographs



Discussion

The discharge in the inlet equals the discharge at the outfall. This shows that an outfall set below the ground surface elevation (submerged) will discharge to the surface under the correct head conditions. This is acceptable for *FCDMC* purposes.

4.2.7.33 Scenario 32: Outfall to Lake Submerged Floodplain with Reverse Flow

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain outfall submerged in a lake. A Type 1 Inlet was used to connect two grid elements. The main setup of the test model consist of (1) a major wash that is 100 feet wide by 10 feet deep, (2) a parallel minor wash 20 feet wide by 8.5 feet deep and (3) the outfall invert is below the *FLO-2D* ground surface elevation to simulate the submerged outfall. A floodplain

Courant value of 0.40 was used to stabilize the model. An inflow hydrograph with a maximum of 7,500 cfs was placed in the major wash with no inflow hydrograph in the minor wash. This test was designed to show that flow travels upstream through the outfall and out the inlet when the water surface elevation at the outfall is greater than at the inlet. A plan view of the test model is shown on Figure 4.209 and Figure 4.210 and a profile view on Figure 4.211.

Figure 4.209 Scenario 32: Plan View of the Test Model – Elevation

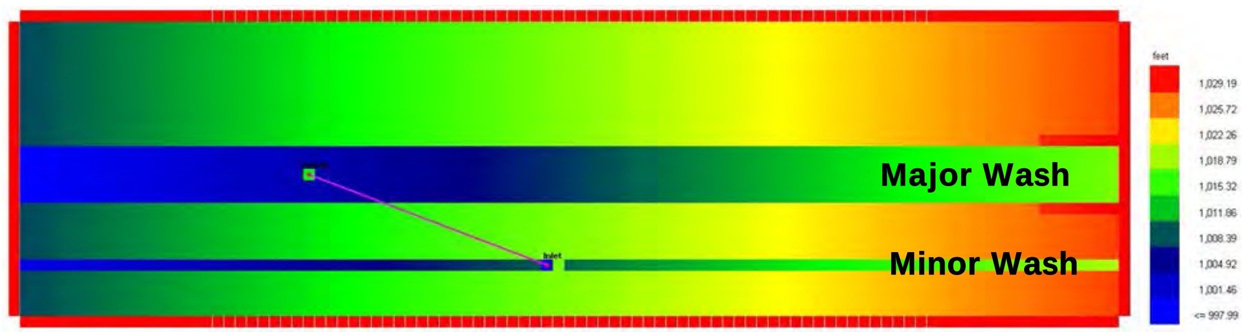


Figure 4.210 Scenario 32: Plan View of the Test Model – Max Depth

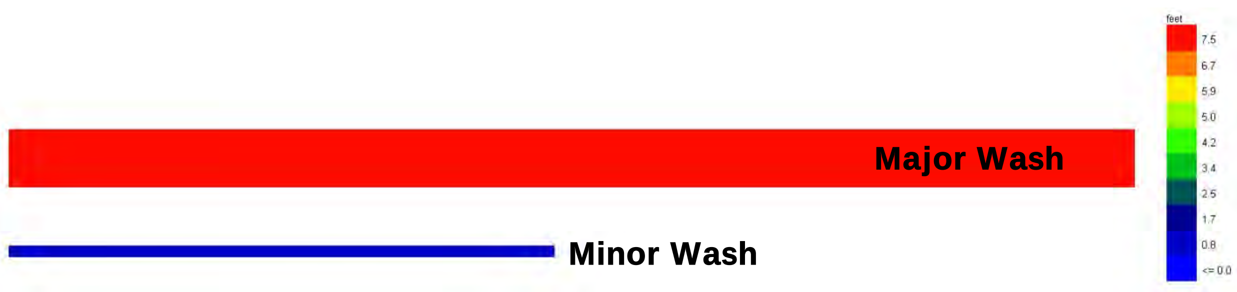
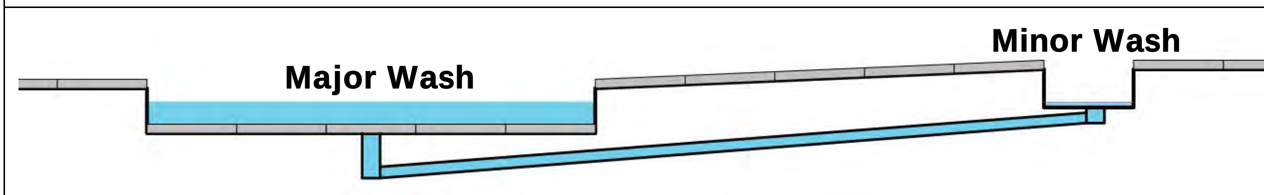


Figure 4.211 Scenario 32: Profile View of the Test Model



<u>Benchmark</u>

At the start of the model, the inlet discharge in *FLO-2D* is equal to zero; there is no initial inflow on the inlet. The *SWMM* outfall water surface elevation is higher than the *SWMM* inlet elevation (*FLO-2D* ground elevation) to force water to flow up the pipe. Once the *SWMM* outfall water surface elevation is higher than the *FLO-2D* ground elevation, there should be water flow leaving the inlet onto the surface causing flow from the major wash into the small wash. This comparison is shown on Figure 4.212 and Figure 4.213.

Figure 4.212 Scenario 32: *SWMM* & *FLO-2D* Hydrographs

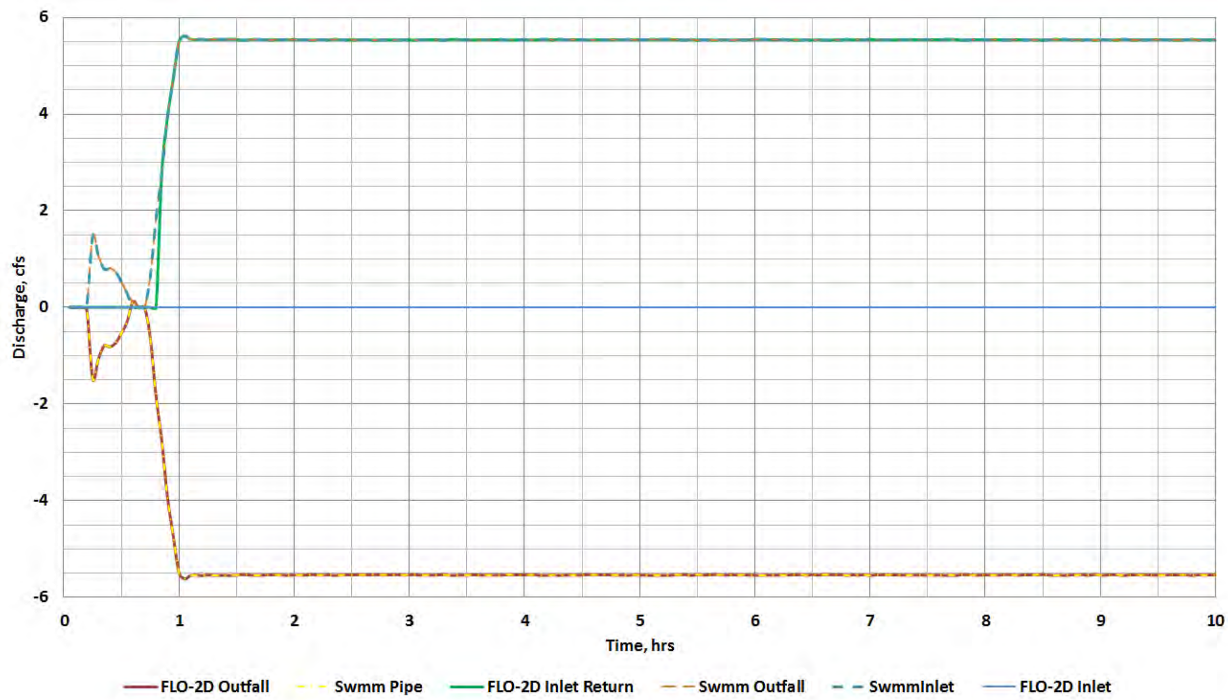
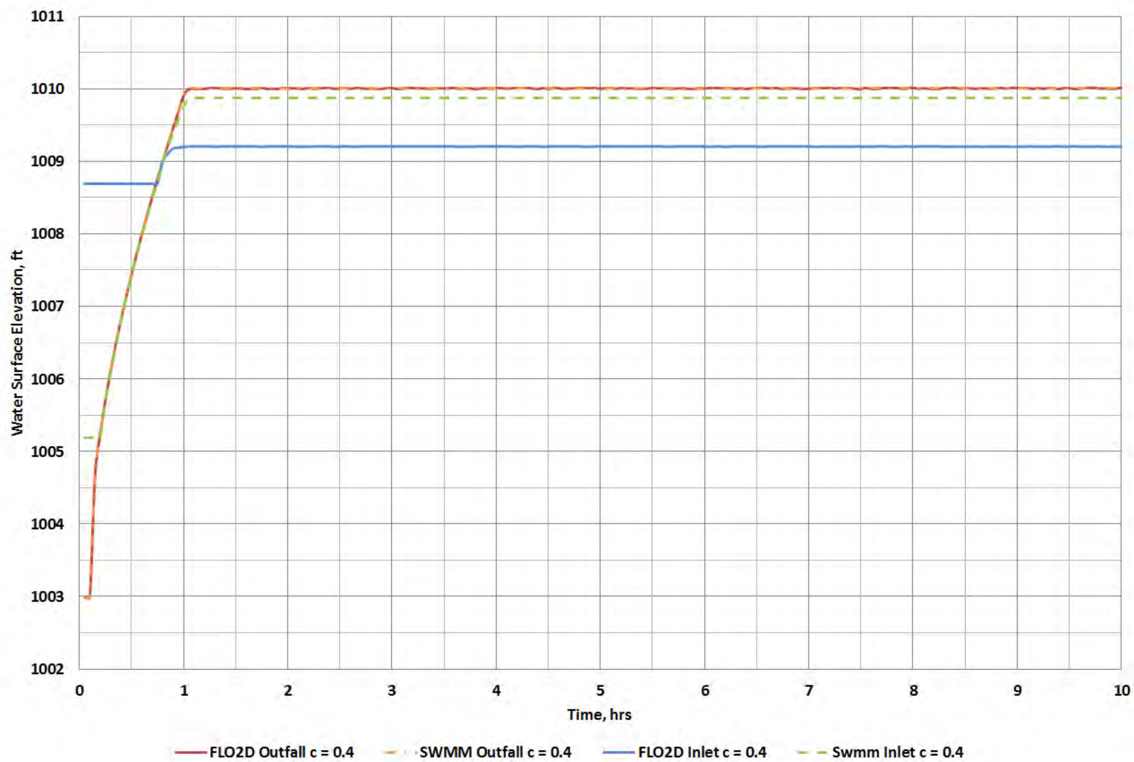


Figure 4.213 Scenario 32: Outfall & Inlet *WSEL*



<u>Discussion</u>

The *SWMM* inlet water surface elevation rises as the *SWMM* outfall water surface elevation rises; causing water to flow upstream in the pipe as shown on Figure 4.213. Once the *SWMM* outfall water surface elevation is above the *FLO-2D* inlet, the *FLO-2D* inlet water surface elevation increases causing water to flow onto the surface. The discharge through the outfall, pipe and inlet are shown on Figure 4.212. Hour 0.2 to .55 represents the pipe filling up between the pipe invert elevation and top of pipe. For this model the outfall water surface elevation is constantly higher than the *FLO-2D* water surface elevation, hence the flow is shown to go upstream through the pipe. The maximum depth in the major and minor washes is shown graphically on Figure 4.210. This is acceptable for *FCDMC* purposes.

4.2.7.34 Scenario 33: Outfall to Lake Submerged *1D*-Channel, Reverse Flow

<u>Scenario</u>

This test is similar to Scenario 32 but the main channel is created using the *1D*-Channel component. The setup of the test model consists of (1) a major wash that is 100 feet wide by 10 feet deep, (2) a parallel minor wash 20 feet wide by 8.5 feet deep and (3) the outfall invert is below the *FLO-2D* ground surface elevation to simulate the submerged outfall. A floodplain Courant value of 0.40 and channel Courant value of 0.20 was used to stabilize the model. An inflow hydrograph with a maximum of 7,500 cfs was placed in the major wash with no inflow hydrograph in the minor wash. This test was designed to show that flow travels upstream in the pipe through the outfall and out the inlet when the water surface elevation at the outfall is greater than at the inlet. A plan view of the test model is shown on Figure 4.214 and Figure 4.215 and a profile view on Figure 4.216.

Figure 4.214 Scenario 33: Plan View of the Test Model – Elevation

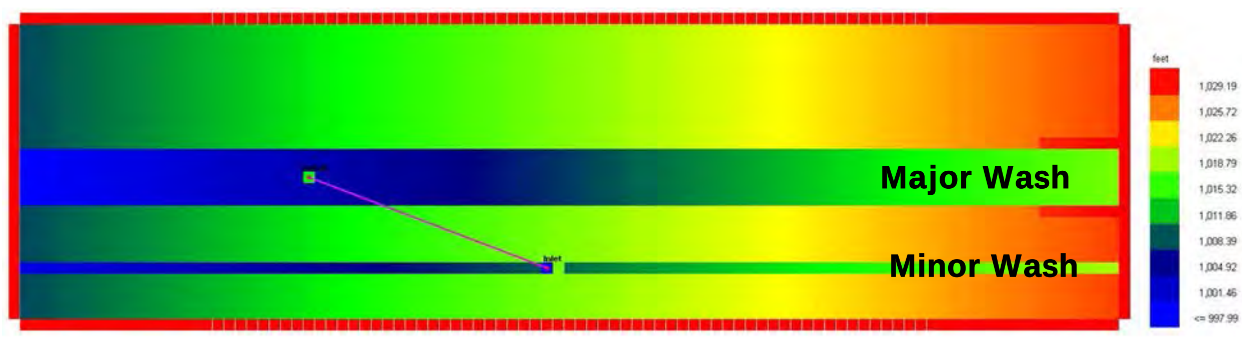
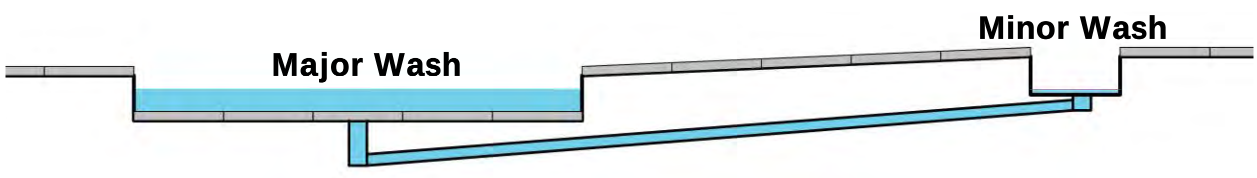


Figure 4.215 Scenario 33: Plan View of the Test Model – Max Depth

Figure 4.216 Scenario 33: Profile View of the Test Model



Benchmark

At the start of the model, the inlet discharge in *FLO-2D* is equal to zero; there is no initial inflow on the inlet. The *SWMM* outfall water surface elevation should be higher than the *SWMM* inlet between invert elevation and the rim elevation (*FLO-2D* ground elevations) to cause water flow up the pipe. Once the *SWMM* outfall water surface elevation is higher than the *FLO-2D* ground elevation, there should be water flow on the surface causing flow from the major wash into small wash through the storm drain inlet. This comparison is shown on Figure 4.217 and Figure 4.218.

Figure 4.217 Scenario 33: *SWMM* & *FLO-2D* Hydrographs

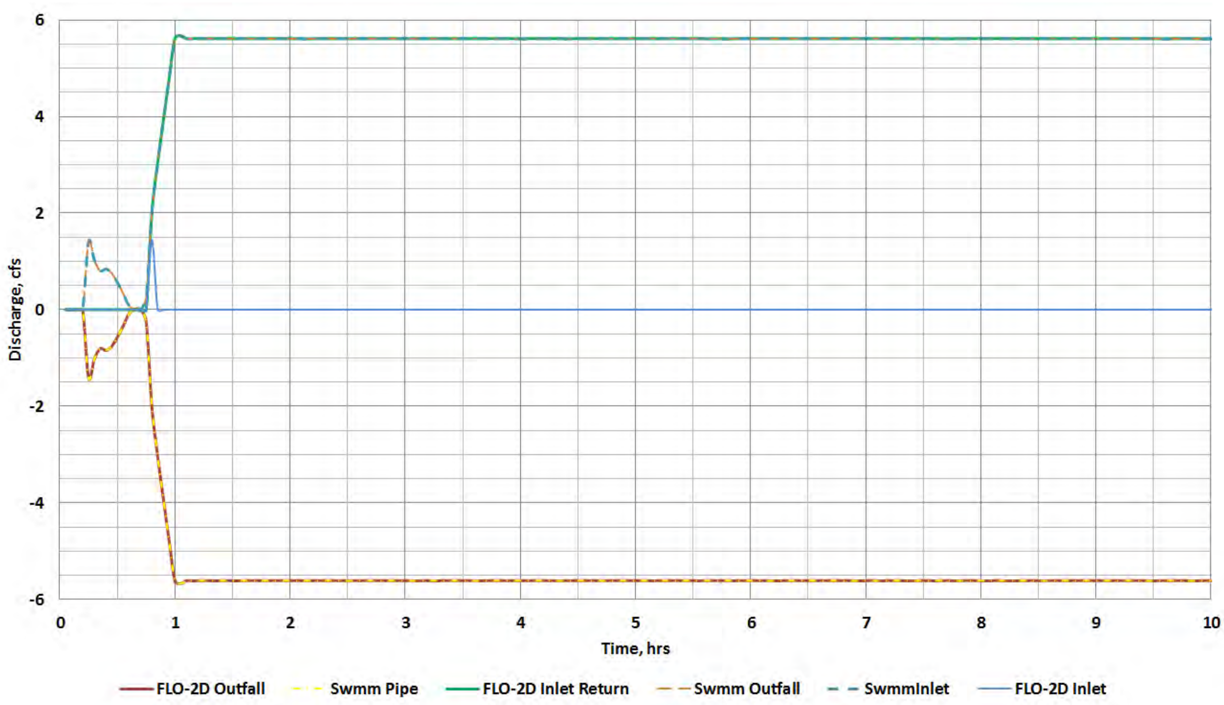
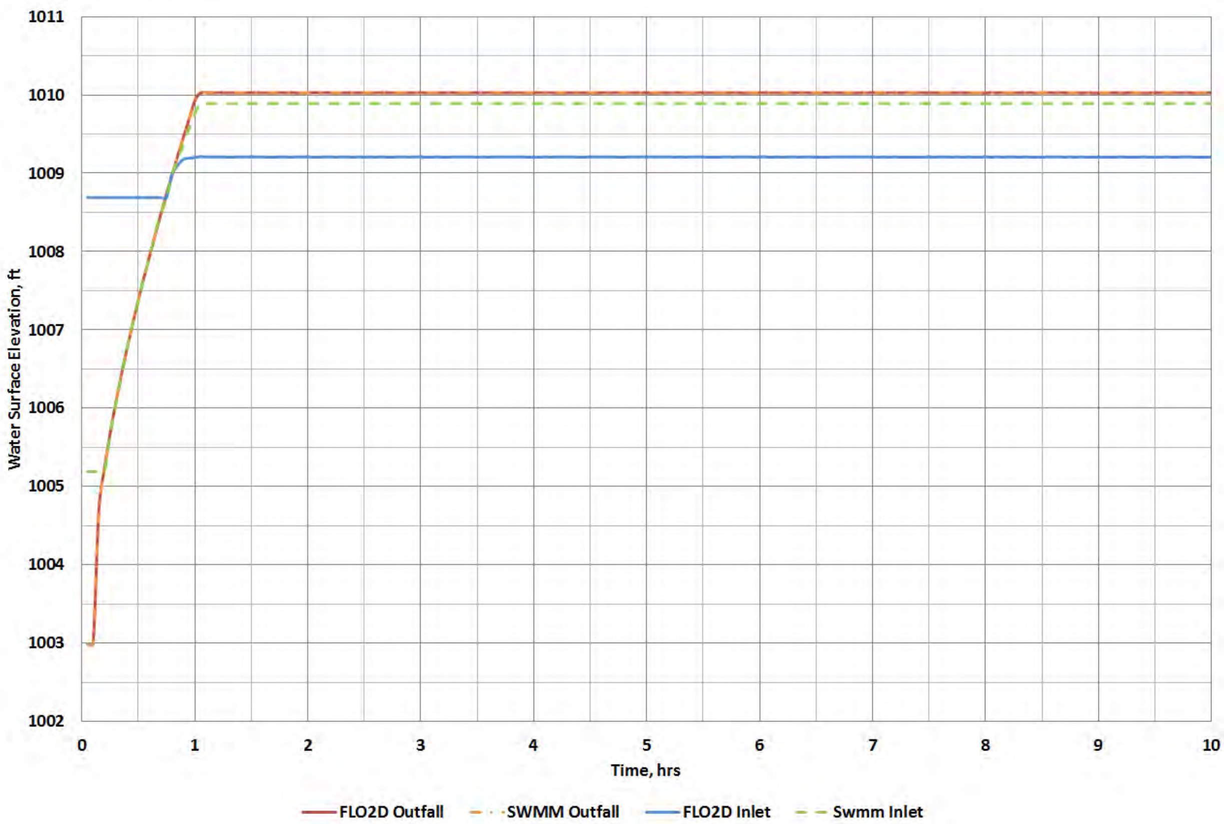


Figure 4.218 Scenario 33: Outfall & Inlet *WSEL*



Discussion

As discussed in Scenario 33, the *SWMM* inlet water surface elevation increases as the *SWMM* outfall water surface elevation increases, causing water flow upstream in the pipe. Refer to Figure 4.218. After the *SWMM* outfall water surface elevation exceeds the *FLO-2D* inlet, flow drains through the inlet and onto the surface in the minor wash. The discharge through the outfall, pipe and inlet are shown on Figure 4.217. Hour 0.2 to 0.55 represents the pipe filling up between the pipe invert elevation and top of pipe. For this model the outfall water surface elevation is constantly higher than the *FLO-2D* water surface elevation, hence the flow is shown move upstream through the pipe. Note that the SWMM inlet WSEL is greater than the FLO-2D inlet WSEL. This represents the head required to force flow through the inlet. The water in the major and minor washes is shown graphically on Figure 4.214. This is acceptable for *FCDMC* purposes.

4.2.7.35 Scenario 34: Outfall to Lake Submerged Floodplain, Flap Gate at Outfall

Scenario

This test uses the same configuration as Scenario 33 with the addition of a flap gate at the pipe outfall; this was coded in the *SWMM*.inp file. This test was created to show that by a adding a flap gate, there will be no backwater effects to the inlet, thus preventing water from entering the pipe. The plan view of the test model is shown on Figure 4.219 and Figure 4.220 and a profile view on Figure 4.221.

Figure 4.219 Scenario 34: Plan View of the Test Model – Elevation

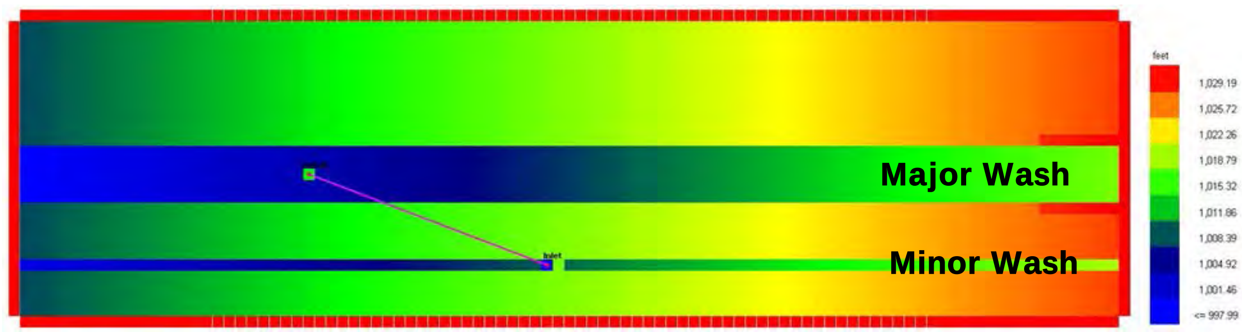
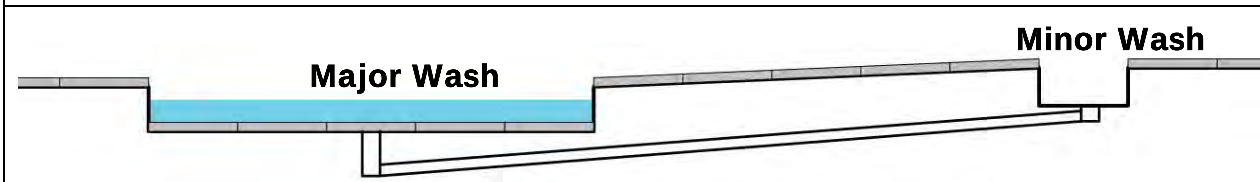


Figure 4.220 Scenario 34: Plan View of the Test Model – Max Depth

Figure 4.221 Scenario 34: Profile View of the Test Model



Benchmark

The inlet, outfall and pipe discharge in *FLO-2D* should be equal to zero; therefore no water should be flowing in the minor wash, only in the major wash. Results are shown on Figure 4.222 and Figure 4.223.

Figure 4.222 Scenario 34: *SWMM* & *FLO-2D* Hydrographs

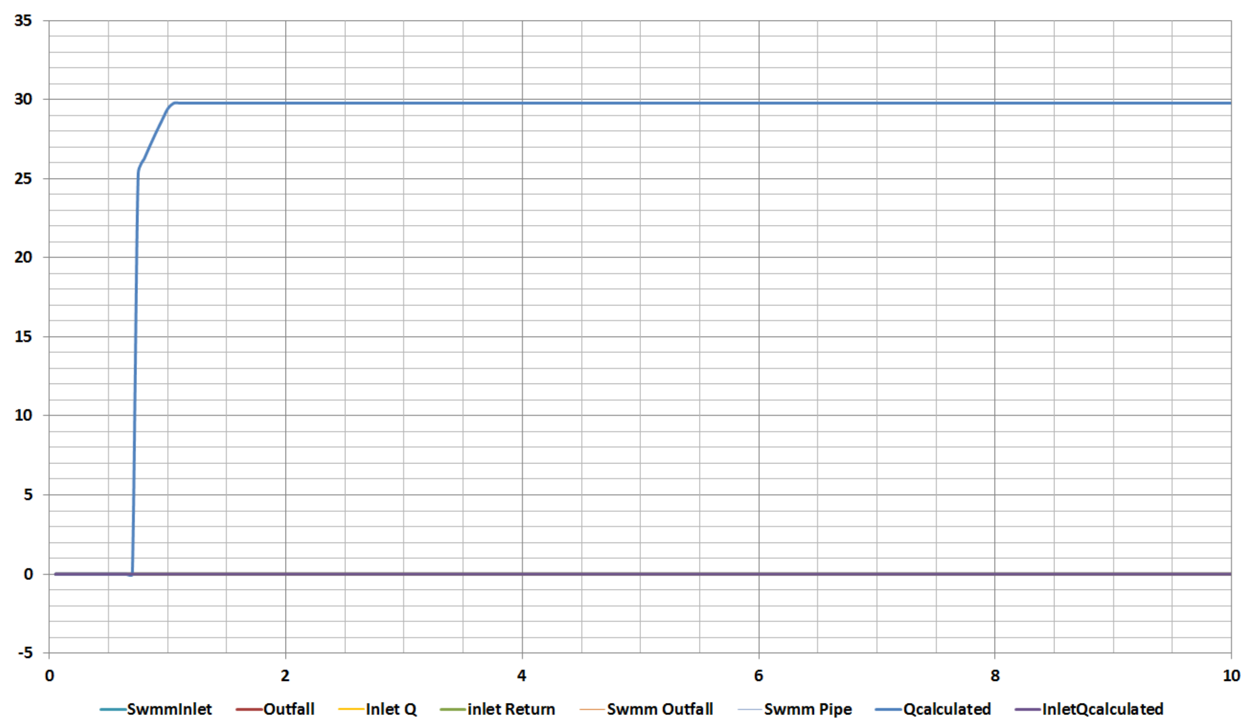
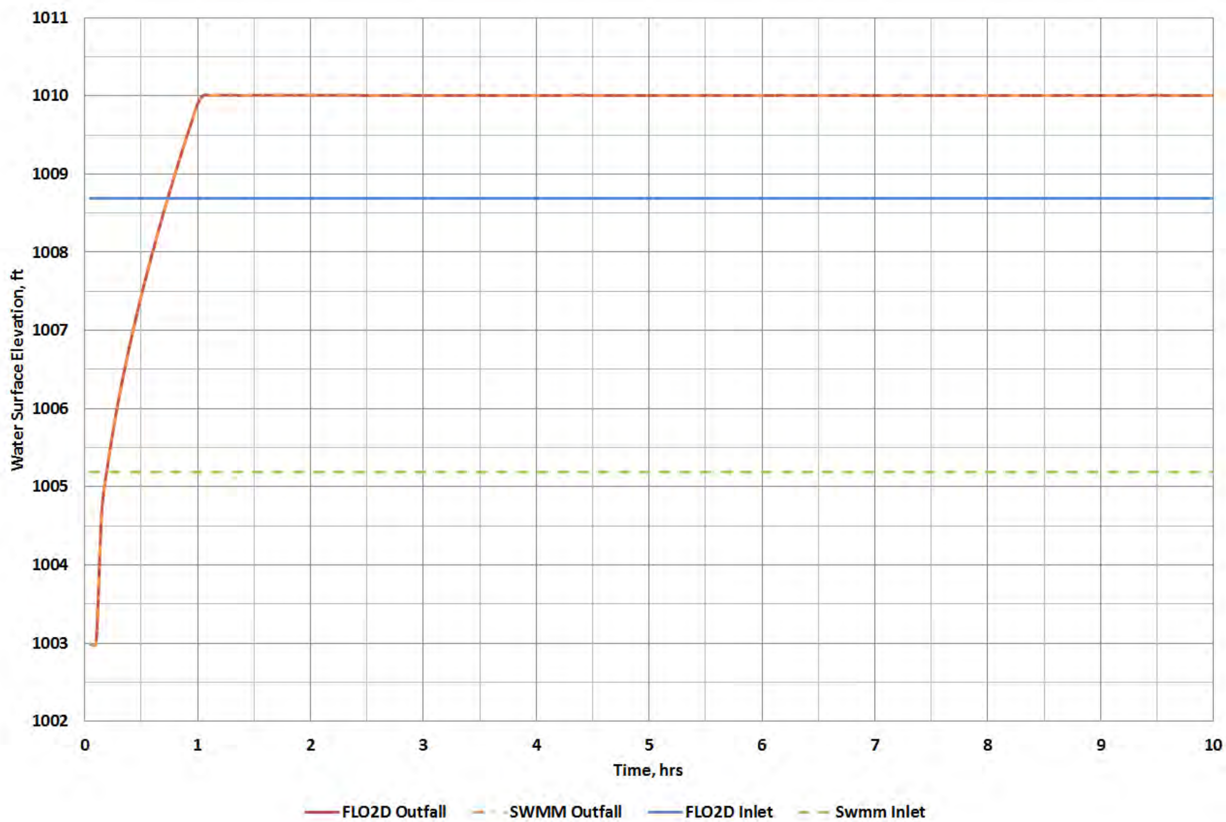


Figure 4.223 Scenario 34: Outfall & Inlet *WSEL*



Discussion

The *SWMM* inlets and *FLO-2D* water surface elevation maintain their initial water surface elevation, showing no water flow going through the storm drain system and this is shown on Figure 4.223. The discharge through outfall, inlet and pipe equal zero, showing no water flow going through the storm drain system and this is shown on Figure 4.222. There is water only in the major wash as expected and is shown graphically on Figure 4.219. The flap gate function in the *SWMM*.inp works as expected. This is acceptable for *FCDMC* purposes.

4.2.7.36 Scenario 35: Outfall to Lake Submerged *1D*-Channel, Flap Date at Outfall

Scenario

This test uses the same set up as Scenario 35 above but with the *1D*-Channel component. This test was created to show that by a adding a flap gate, there will be no backwater effects to the inlet, thus preventing water from entering the pipe. A plan view of the test model is shown on Figure 4.224 and Figure 4.225 and a profile view on Figure 4.226.

Figure 4.224 Scenario 35: Plan View of the Test Model – Elevation

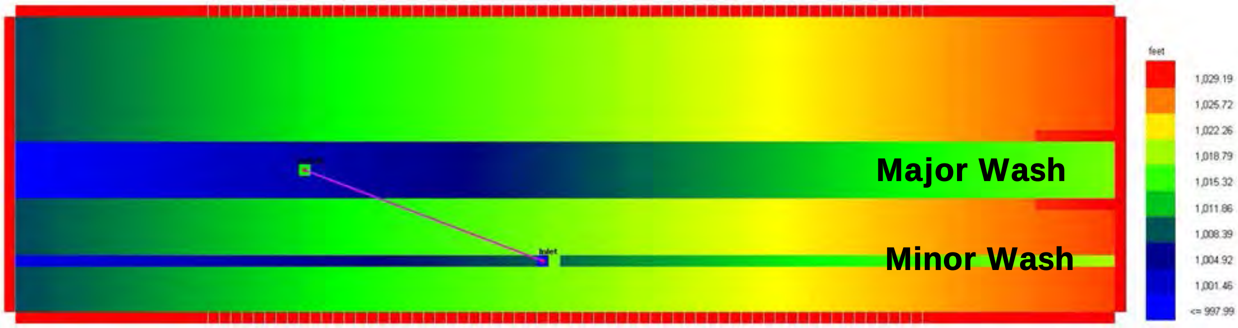
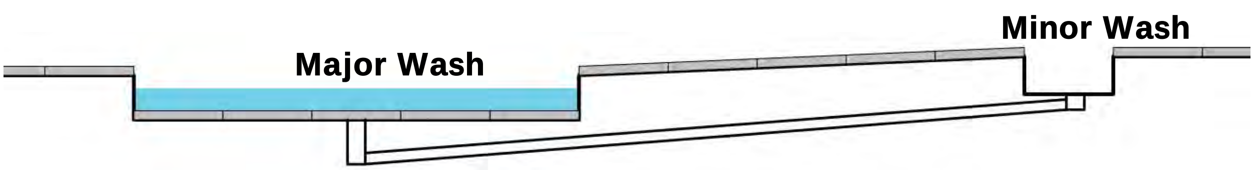


Figure 4.225 Scenario 35: Plan View of the Test Model – Max Depth

Figure 4.226 Scenario 35: Profile View of the Test Model



Benchmark

The inlet, outfall and pipe discharge in *FLO-2D* should be equal to zero; therefore no water should be flowing in the minor wash, only in the major wash. Results are shown on Figure 4.227 and Figure 4.228.

Figure 4.227 Scenario 35: *SWMM* & *FLO-2D* Hydrographs

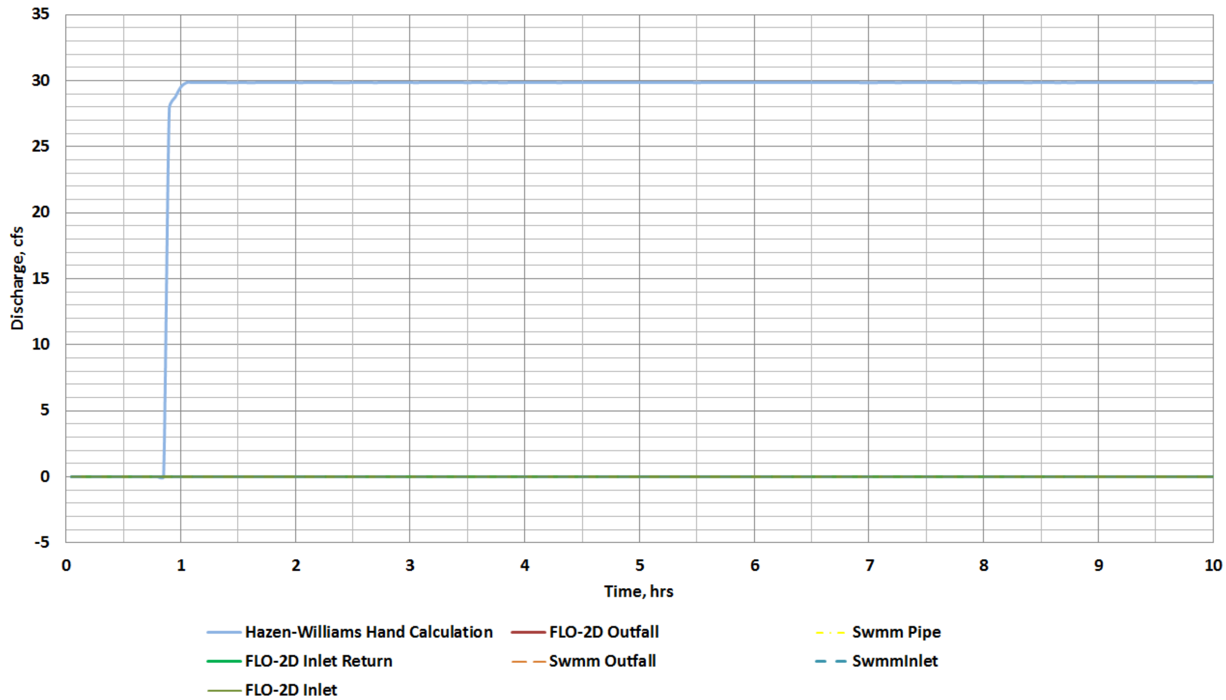
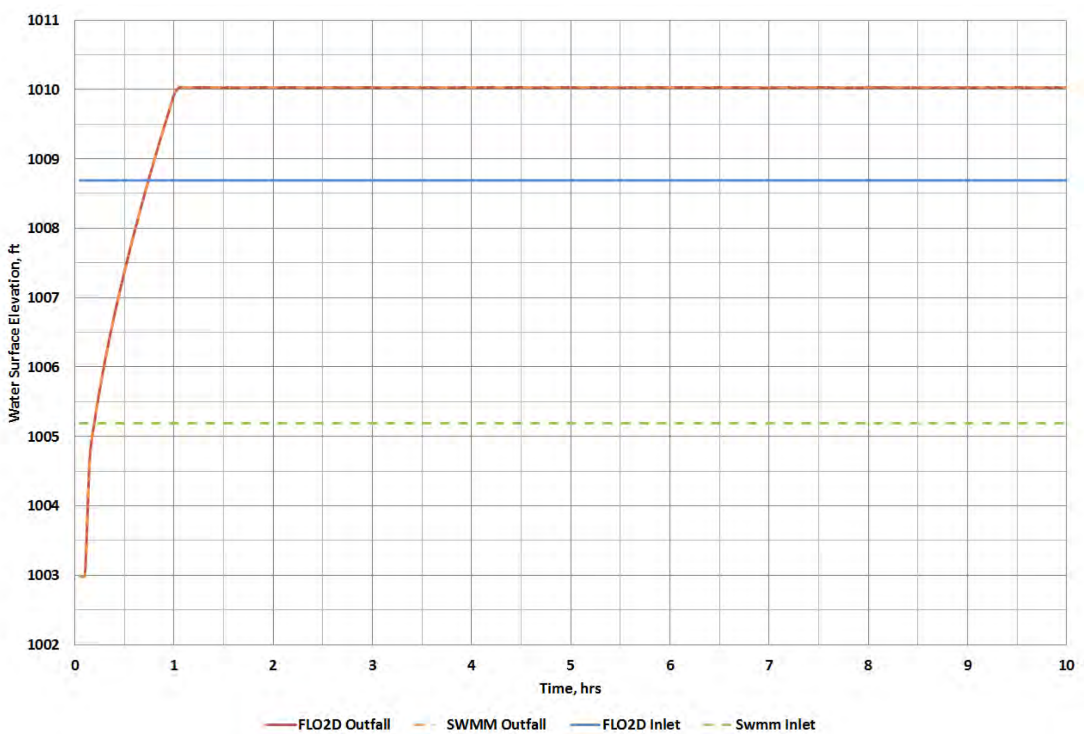


Figure 4.228 Scenario 35: Outfall & Inlet *WSEL*



Discussion

As discussed in Scenario 35, Figure 4.228 shows the *SWMM* inlets and *FLO-2D* water surface elevation maintain their initial water surface elevation, showing no water flow going through the storm drain system. The discharge through outfall, inlet and pipe equal to zero, showing no water flow going through the storm drain system is shown on Figure 4.227. There is water only in the major wash as expected and is shown graphically on Figure 4.224. The flap gate function in the *SWMM*.inp works as expected. This is acceptable for *FCDMC* purposes.

4.2.7.37 Scenario 36: Outfall to Lake Submerged Floodplain, Test Low and High Flows

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain outfall submerged in a lake. The main setup of the test model consist of (1) a major wash that is 100 feet wide by 10 feet deep, (2) a parallel minor wash 20 feet wide by 8.5 feet deep, (3) a storm connecting the minor channel to the main channel with a Type 1 inlet at the upstream end, and (4) an outfall into the main channel with the invert below the *FLO-2D* ground surface elevation simulating a submerged outfall. An inflow hydrograph with an initial maximum discharge of 7,500 cfs and a low discharge of 4,000 cfs is placed in the major wash and a constant 20 cfs inflow hydrograph in the minor wash. The floodplain Courant was changed to 0.2 to stabilize the model. This test was designed to show different head conditions cause water to change flow direction in the pipe. Plan views are shown on Figure 4.229 and Figure 4.230 and a profile view on Figure 4.231.

Figure 4.229 Scenario 36: Plan View of the Test Model – Elevation

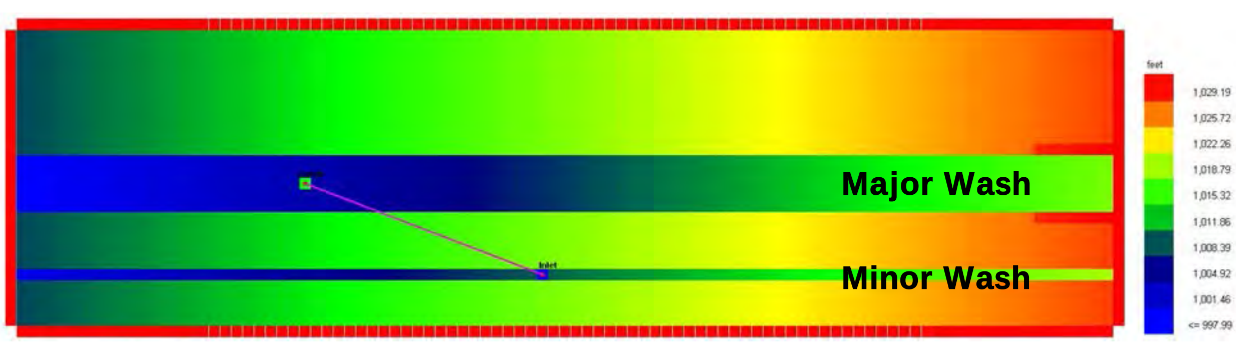


Figure 4.230 Scenario 36: Plan View of the Test Model – Max Depth

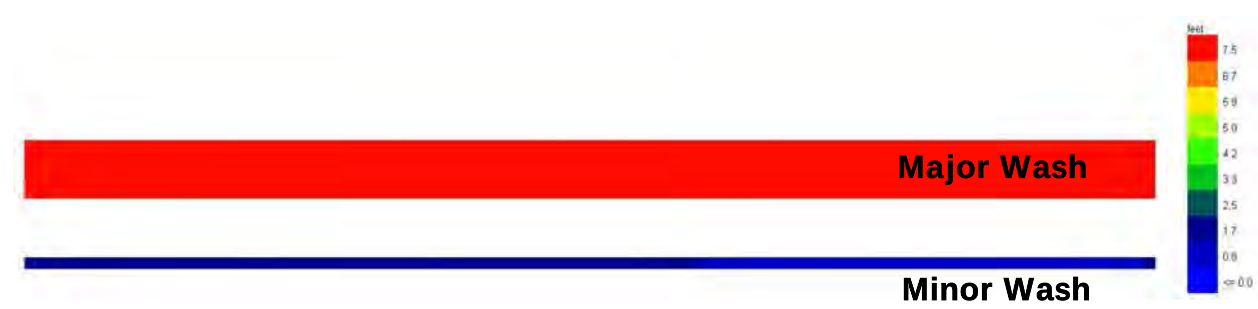
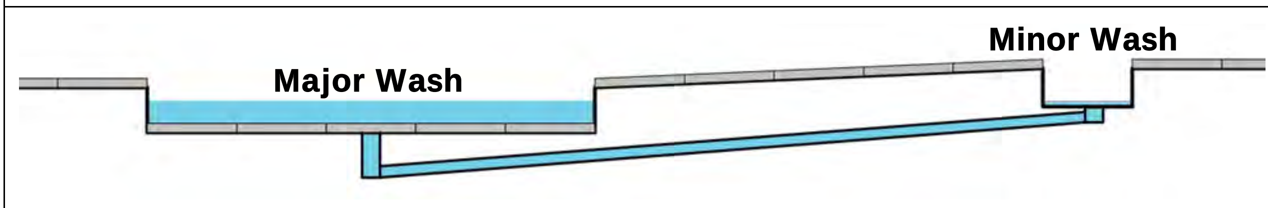


Figure 4.231 Scenario 36: Profile View of the Test Model



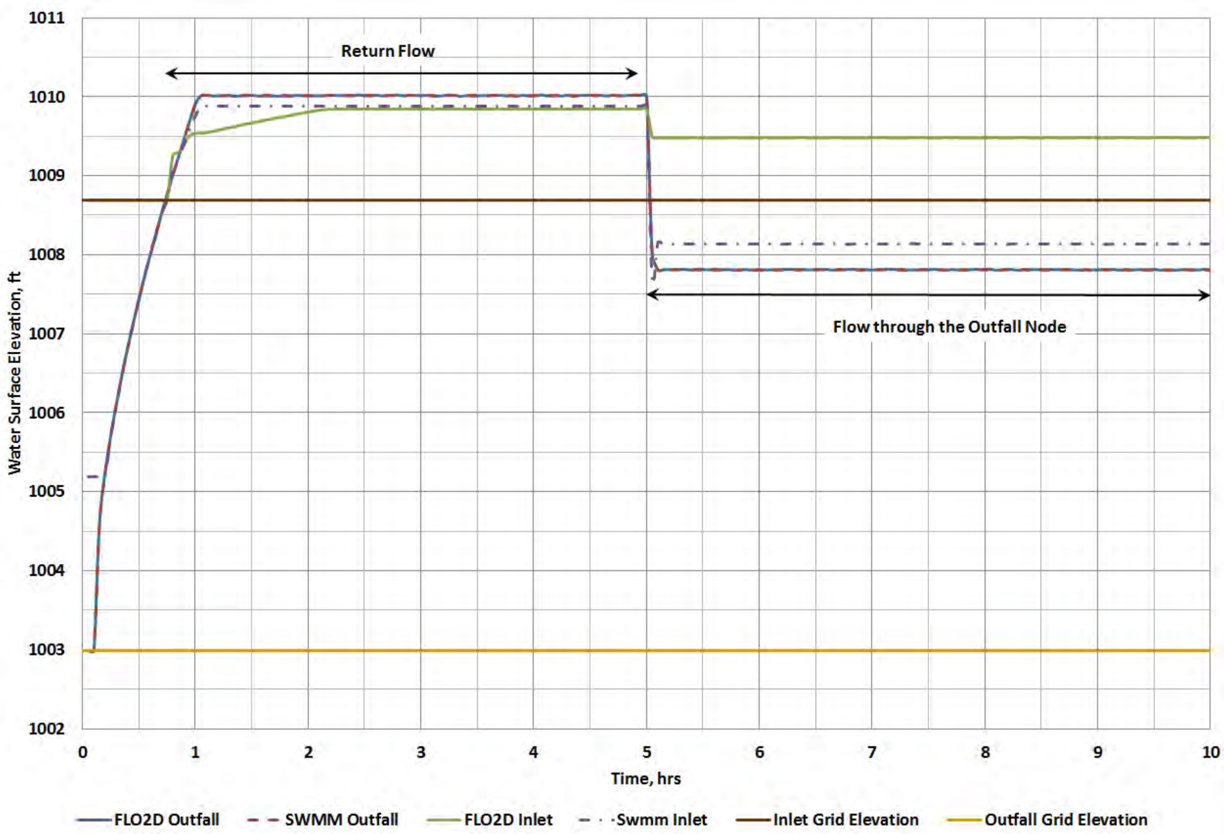
<u>Benchmark</u>

The major wash is forced to have a higher water surface elevation at the beginning of the run compared to the minor wash; this should force water backwards up the pipe. The water surface is then forced to be lower in the major wash compared to the minor wash; this should force water from the minor wash through the pipe into the major wash. During a period of transition where the water surface elevations in both the minor and major wash are equal, there should not be any water flowing through the pipe. This comparison is shown on Figure 4.232.

<u>Discussion</u>

When the water surface elevation in the major wash is higher than the minor wash and the *SWMM* water surface elevation in the pipe is higher than the *FLO-2D* inlet water surface elevation, water will move backwards into the pipe and out the inlet (return flow). The amount of water that is discharged is dependent on the storage volume of the standpipe and the head difference between the *SWMM* pressure head and the *FLO-2D* inlet head. Many of these values are internal to *FLO-2D* and *SWMM* and an accurate hand calculation check of return flow is not possible. Instead the reported *FLO-2D* water surface elevations on the inlet and outfall grids are compared to the SWMM inlet and outfall head elevations (see Figure 4.232). The *SWMM* inlet and *FLO-2D* inlet elevations match and the *SWMM* Outfall and *FLO-2D* outfall elevations match. This scenario functions as expected. This is acceptable for *FCDMC* purposes.

Figure 4.232 Scenario 36: Outfall & Inlet *WSEL*



4.2.7.38 Scenario 37: Outfall to Lake Submerged *1D*-Channel, Low and High Flows

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain outfall submerged in a lake, a Type 1 Inlet was used to connect two grid elements. The main setup of the test model consist of (1) a major wash that is 100 feet wide by 10 feet deep, (2) a parallel minor wash 20 feet wide by 8.5 feet deep and (3) the outfall invert is below the *FLO-2D* ground surface elevation simulating a submerged outfall. An inflow hydrograph with an initial max of 7500 cfs and low of 4000 cfs was placed in the major wash and constant 20 cfs inflow hydrograph in the minor wash. The floodplain Courant of 0.20 and *1D*-Channel Courant of 0.60 was used to stabilize the model. This test was designed to show different head conditions cause water to change flow direction in the pipe. Plan views of the test model are shown on Figure 4.233 and Figure 4.234 and a profile view on Figure 4.235.

Figure 4.233 Scenario 37: Plan View of the Test Model – Elevation

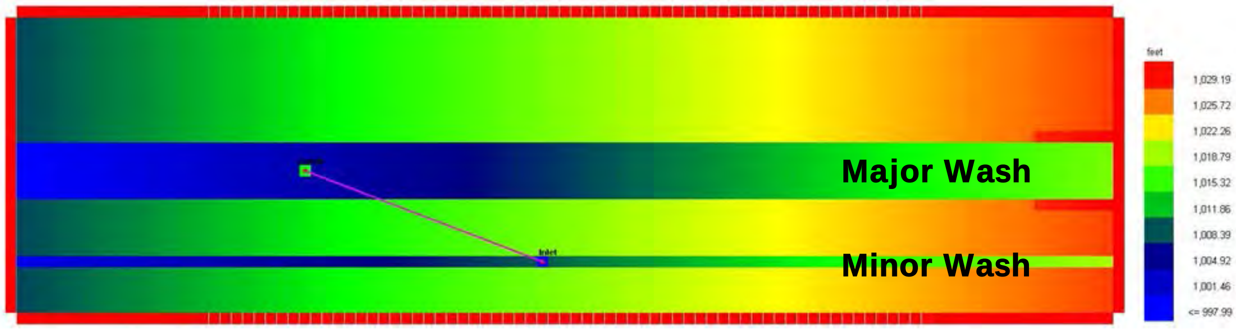
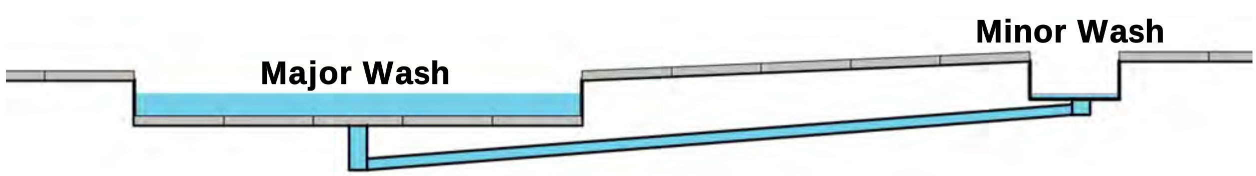


Figure 4.234 Scenario 37: Plan View of the Test Model – Max Depth

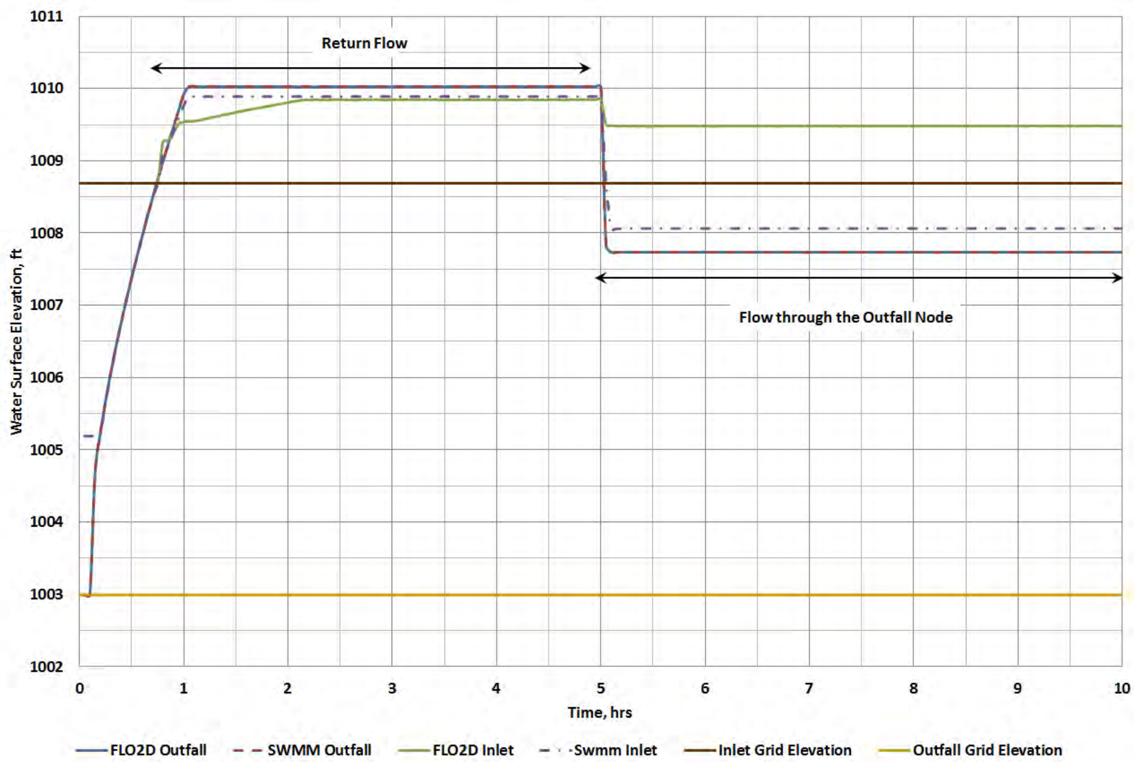
Figure 4.235 Scenario 37: Profile View of the Test Model



<u>Benchmark</u>

The major wash should have a higher water surface elevation at the beginning of the run compared to the minor wash; this should force water backwards up the pipe. The water surface should then be lower in the major wash compared to the minor wash; this should force water through the pipe into the major wash. During a period of transition where the water surface elevation in both minor and major wash, there should not be any water flowing through the pipe, therefore the discharge should be equal to zero. This comparison is shown on Figure 4.236.

Figure 4.236 Scenario 37: Outfall & Inlet *WSEL*



Discussion

Initially, the water surface elevation in the major wash is higher than the minor wash and the *SWMM* outfall water surface elevation is higher than the *FLO-2D* Inlet water surface elevation, forcing water backwards into the pipe and out the inlet (return flow). The *SWMM* inlet and *FLO-2D* inlet elevations match as well as the *SWMM* outfall and *FLO-2D* outfall elevations match (see Figure 4.236). The scenario is then reversed, forcing water from the minor channel into the major channel. This scenario functions as expected. This is acceptable for *FCDMC* purposes.

4.2.7.39 Scenario 38: Outfall to Floodplain, Tailwater below Critical Depth in Pipe

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain pipe that drains from a minor wash into a major wash. The upstream end has a Type 1 Inlet: Curb Opening Inlet at Grade operating in a weir flow condition. The storm drain discharges into a main channel using an outfall. The intent is to test a scenario where flow exiting the storm drain at the outfall has a tailwater depth below critical depth in the pipe. The main setup of the test model consists of (1) a major wash that is 100 feet wide by 10 feet deep,

(2) a parallel minor wash 20 feet wide by 8.5 feet deep and (3) the outfall invert is at the bottom of the *FLO-2D* ground surface elevation. An inflow hydrograph with a maximum discharge of 5 cfs was applied to the minor wash and a constant flow hydrograph of 270 cfs in the major wash. A uniform discharge of 270 cfs in the major wash equates to 0.96 feet of water depth. According to the *SWMM* manual, when using a FREE outfall, the outfall stage is determined by the smaller of the critical flow depth in the conduit or normal flow depth in the outfall channel. This model was created to test if the pipe's critical depth or the channel *WSEL* would be used for the boundary condition in the pipe. For this scenario, the *WSEL* in the major channel is lower than the critical depth *WSEL* in the pipe. Using Flow Master, the pipes critical depth was identified to be 1.16 feet. A floodplain Courant value of 0.20 was used to stabilize the model. Plan views are shown on Figure 4.237 and Figure 4.238 and a profile view on Figure 4.239.

Figure 4.237 Scenario 38: Plan View of the Test Model - Elevation

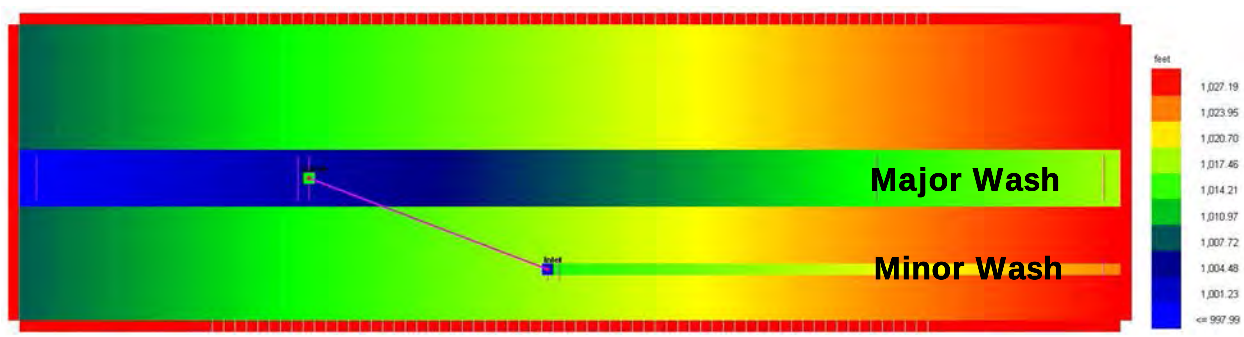
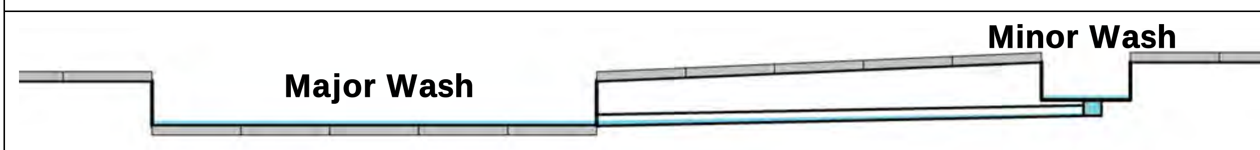


Figure 4.238 Scenario 38: Plan View of the Test Model – Max Depth

Figure 4.239 Scenario 38: Profile View of the Test Model



Benchmark

The *FLO-2D* inlet discharge and a hand-calculated verification discharge are shown on Figure 4.240 to verify that the inlet is receiving 5 cfs. A comparison of water surface elevations at the inlet and outfall are shown on Figure 4.241.

Figure 4.240 Scenario 38: Inflow Hydrographs

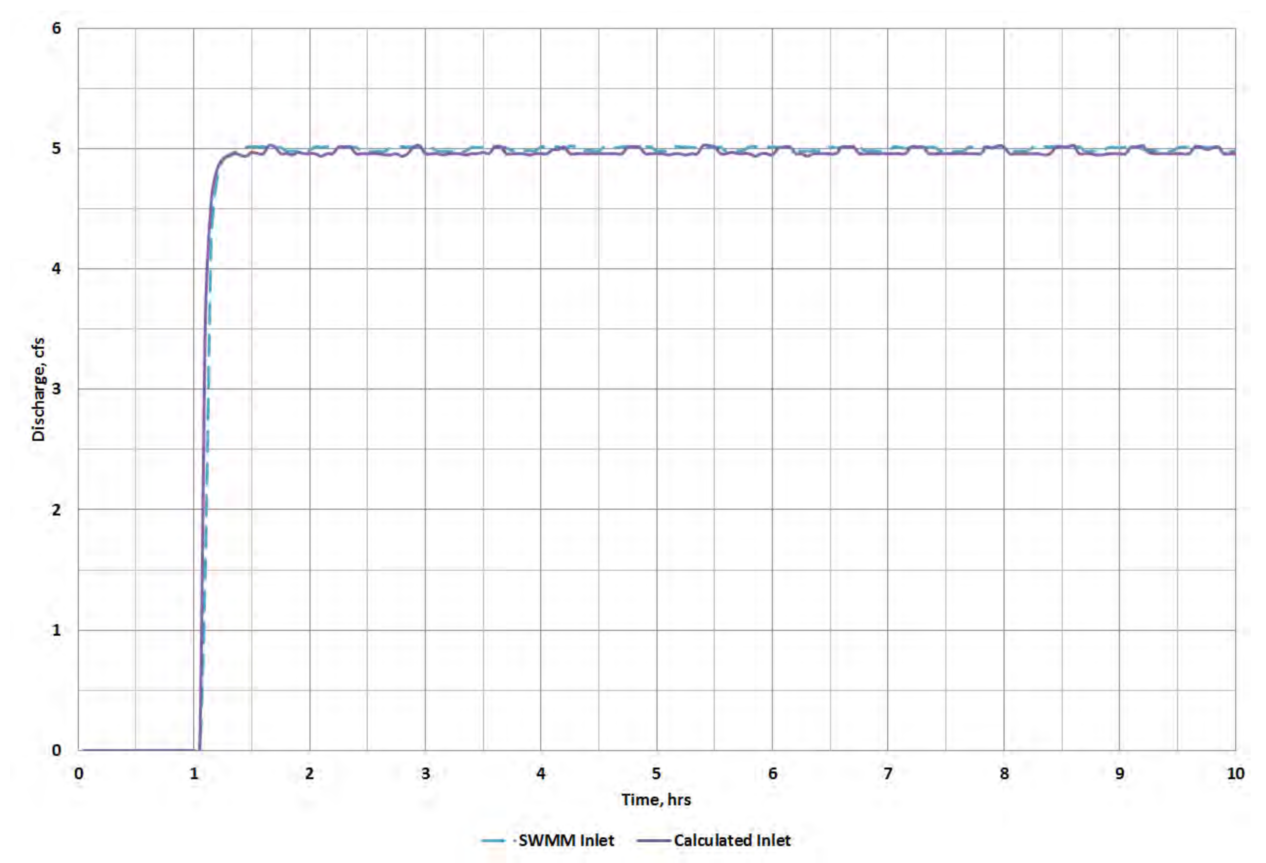
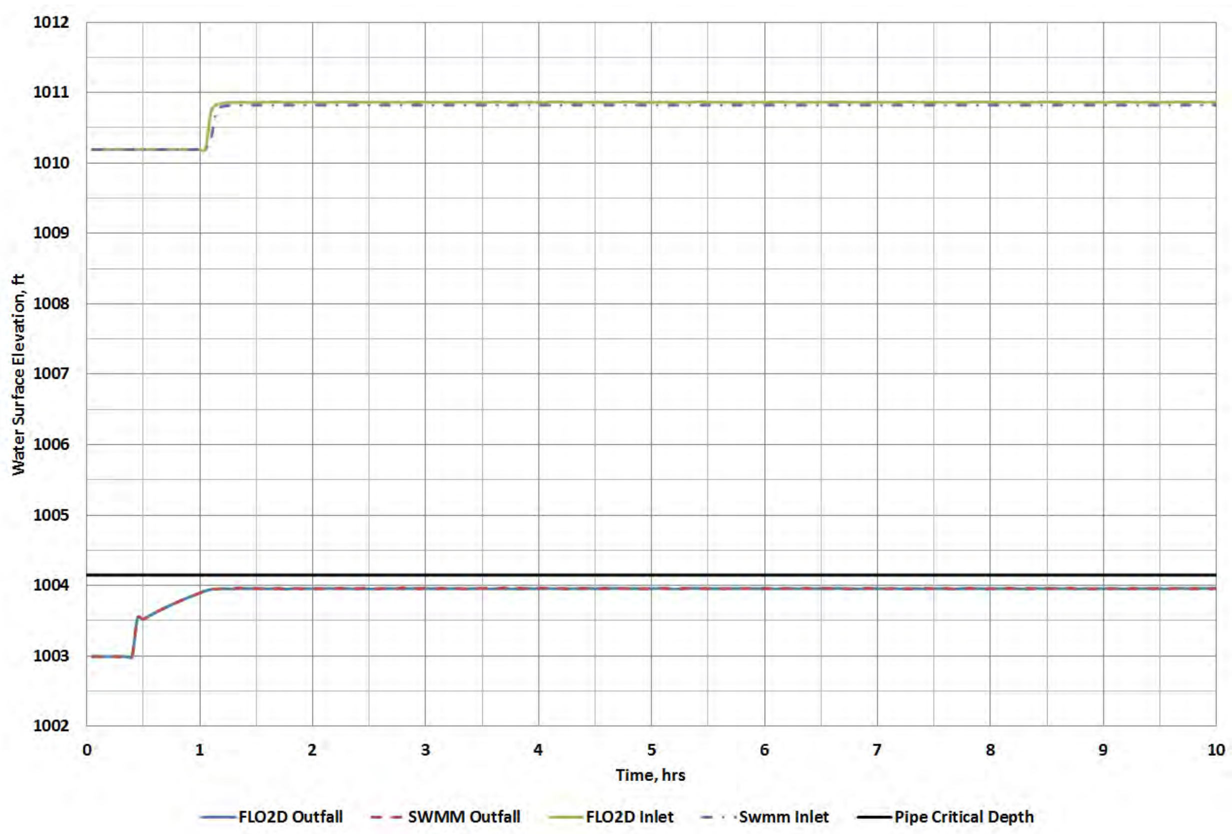


Figure 4.241 Scenario 38: Outfall & Inlet *WSEL*



Discussion

The inlet is receiving 5 cfs as shown on Figure 4.240. The *SWMM* outfall water surface elevation in the pipe at the outfall is the same as the water surface elevation in the major wash as shown on Figure 4.241. The *SWMM* outfall water surface elevation is lower than the pipe's critical depth *WSEL*, therefore the pipe boundary condition is based on the *WSEL* in the channel. This is acceptable for *FCDMC* purposes.

4.2.7.40 Scenario 39: Outfall to *1D*-Channel, Tailwater below Critical Depth in Pipe

Scenario

The main setup of the test model is the same as Scenario 38, the only difference is the *1D*-channel component is being used to model the major wash. An inflow hydrograph with a maximum discharge of 5 cfs was applied to the minor wash and a uniform flow hydrograph of 270 cfs in the major wash. The 270 cfs discharge in the major wash equates to 0.96 feet water depth. This model was created to test if the pipe's critical depth or the channel *WSEL* would be used for the boundary condition in the pipe.

Using Flow Master, the pipes critical depth was identified to be 1.16 feet. A floodplain Courant value of 0.60 and *1D*-channel Courant value of 0.20 was used to stabilize the model. Plan views are shown on Figure 4.242 and Figure 4.243 and a profile view on Figure 4.244.

Figure 4.242 Scenario 39: Plan View of the Test Model - Elevation

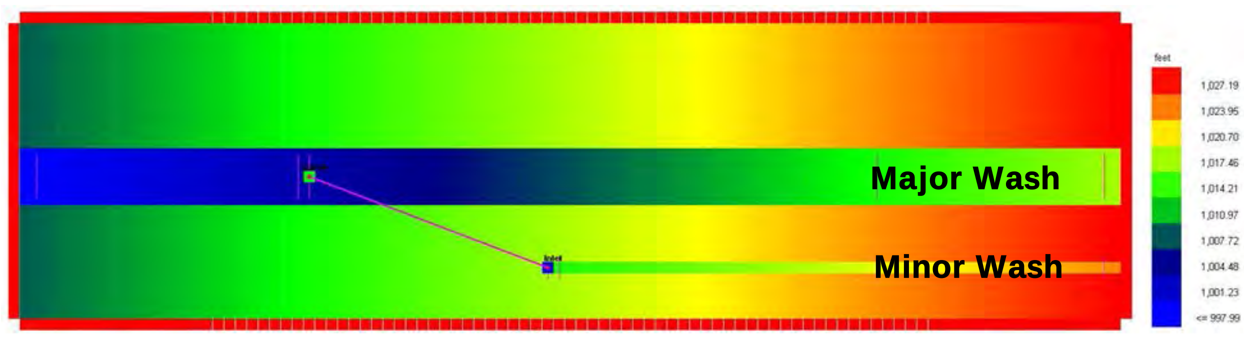
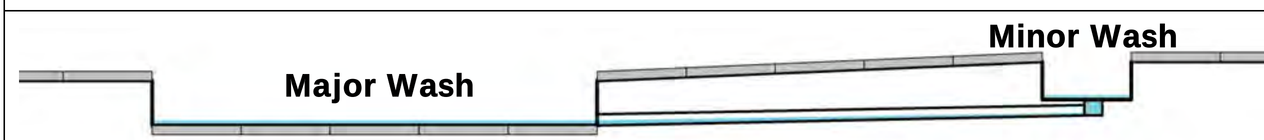


Figure 4.243 Scenario 39: Plan View of the Test Model – Max Depth

Figure 4.244 Scenario 39: Profile View of the Test Model



Benchmark

The *FLO-2D* inlet hydrograph and a hand-calculated verification hydrograph are shown on Figure 4.245 to verify that the inlet is receiving 5 cfs. A comparison of water surface elevations for both the inlet and outfall are shown on Figure 4.246.

Figure 4.245 Scenario 39: Inflow Hydrographs

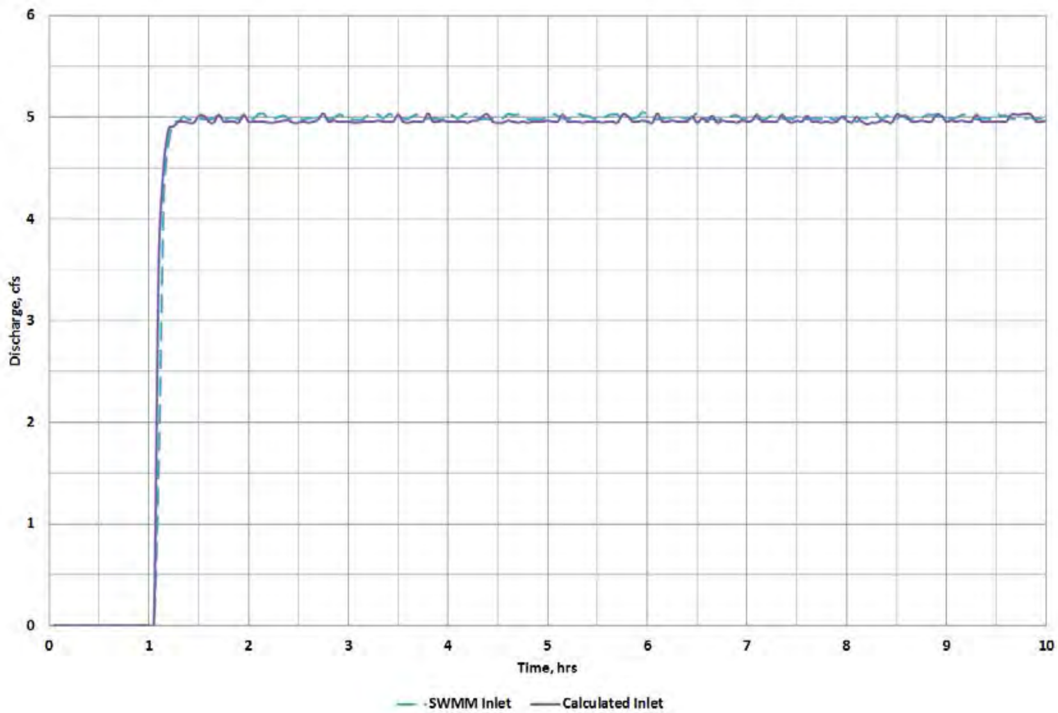
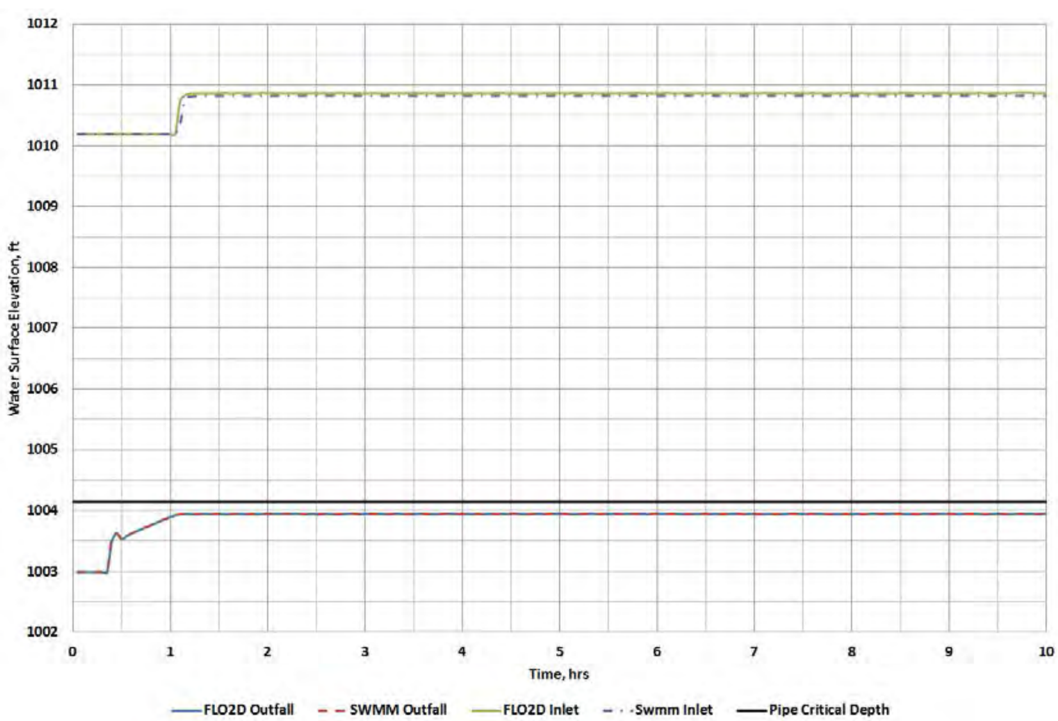


Figure 4.246 Scenario 39: Outfall & Inlet *WSEL*



Discussion

The inlet is receiving 5 cfs as shown on Figure 4.250. The *SWMM* outfall water surface elevation in the pipe at the outfall is the same as the *WSEL* in the major wash as shown on Figure 4.241. The *SWMM* outfall *WSEL* is lower than the pipe's critical depth *WSEL*, therefore the pipe boundary condition is based on the *WSEL* in the channel. This is acceptable for *FCDMC* purposes.

4.2.7.41 Scenario 40: Outfall to Floodplain, Tailwater below Soffit, above Crit. Depth

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain pipe that drains from a minor wash into a major wash. The upstream end has a Type 1 Inlet: Curb Opening Inlet at Grade operating in a weir flow condition. The storm drain discharges into a main channel using an outfall. The intent is to test a scenario where flow exiting the storm drain at the outfall has a tailwater depth above critical depth in the pipe but below the pipe soffit elevation. The main setup of the test model consist of (1) a major wash that is 100 feet wide by 10 feet deep, (2) a parallel minor wash 20 feet wide by 8.5 feet deep and (3) the outfall invert is at the bottom of the *FLO-2D* ground surface elevation. An inflow hydrograph with a maximum discharge of 5 cfs is applied to the minor wash and a uniform discharge of 530 cfs in the major wash. The 530 cfs discharge in the major wash equates to 1.45 feet of water depth. This model was created to test if the pipe's critical depth *WSEL* or the main channel *WSEL* would be used for the boundary condition at the storm drain outfall. Using Flow Master, the pipes critical depth was identified to be 1.16 feet. A floodplain Courant value of 0.20 was used to stabilize the model. Plan views of the test model are shown on Figure 4.247 and Figure 4.248 and the profile view is on Figure 4.249.

Figure 4.247 Scenario 40: Plan View of Test Model - Elevation

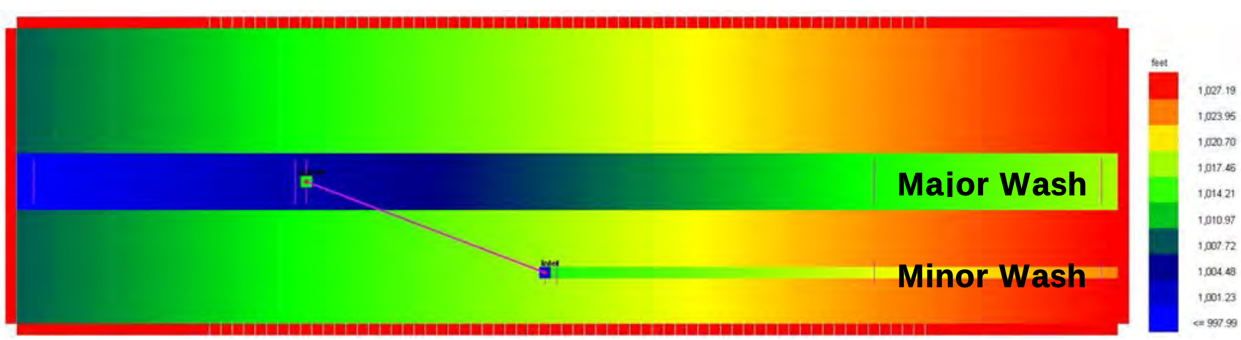


Figure 4.248 Scenario 40: Plan View of the Test Model – Max Depth

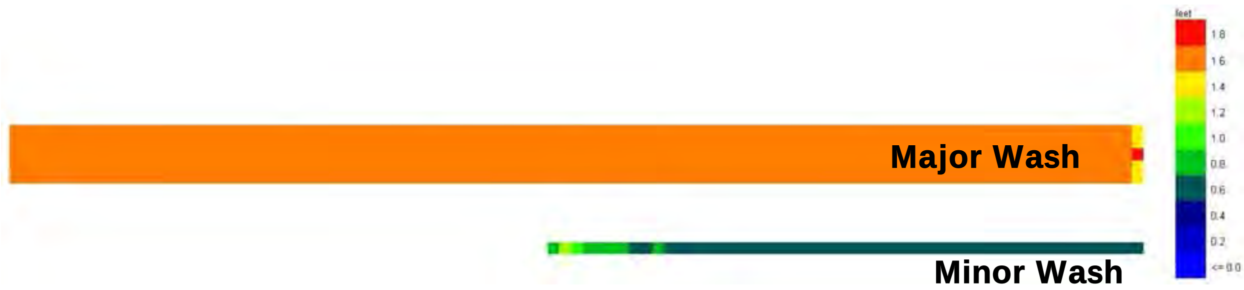
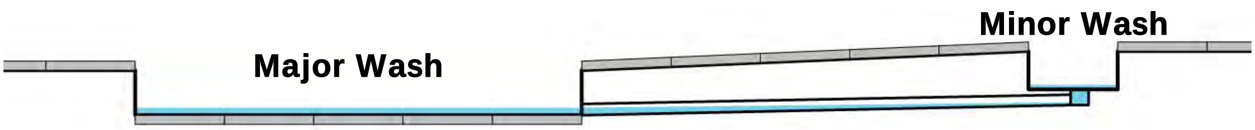


Figure 4.249 Scenario 40: Profile View of the Test Model



Benchmark

The *FLO-2D* inlet hydrograph and the results of hand-calculated verification hydrograph are shown on Figure 4.250 to verify that the inlet is receiving 5 cfs appropriately. A comparison of water surface elevations at the inlet and outfall are shown on Figure 4.251.

Figure 4.250 Scenario 40: Inflow Hydrographs

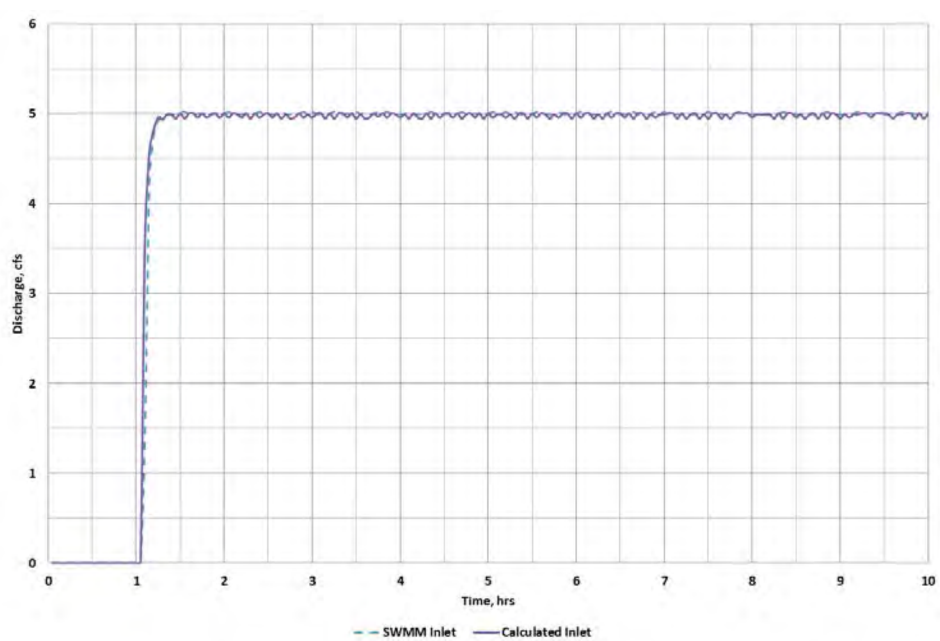
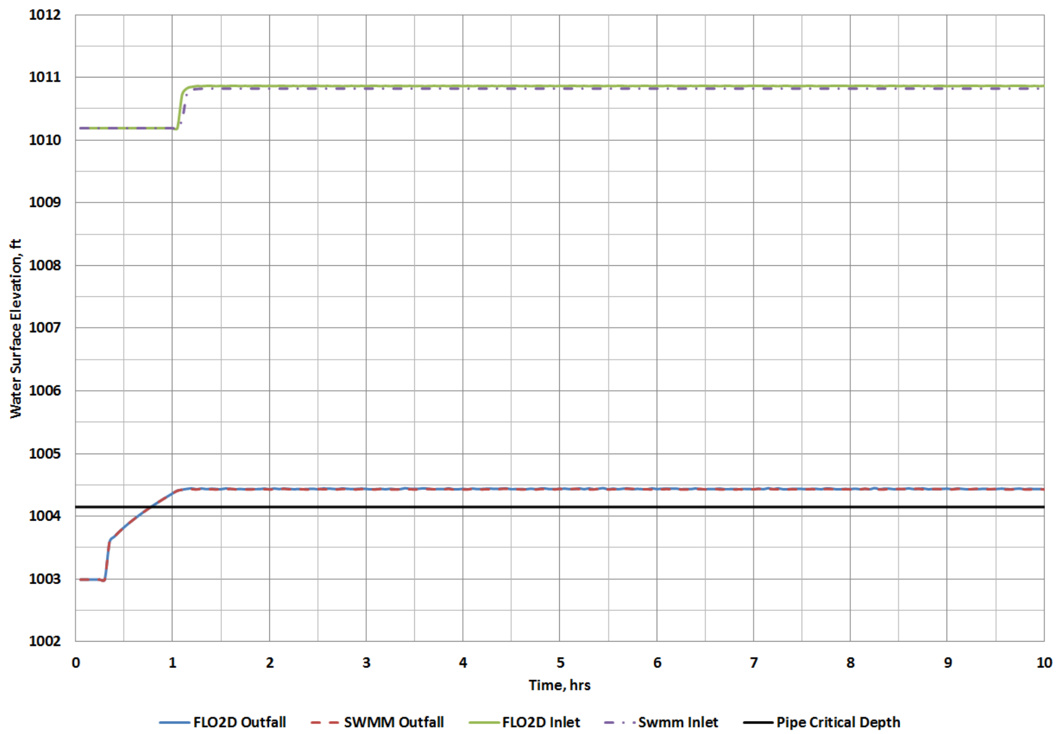


Figure 4.251 Scenario 40: Outfall & Inlet WSE



Discussion

The inlet is receiving 5 cfs as shown on Figure 4.250. The *SWMM* outfall WSEL in the pipe at the outfall is the same as the *WSEL* in the major wash as shown on Figure 4.251. The *SWMM* outfall *WSEL* is higher than the pipe's critical depth *WSEL*, therefore the pipe boundary condition is based on the *WSEL* in the channel. This is acceptable for *FCDMC* purposes.

4.2.7.42 Scenario 41: Outfall to *1D*-Channel, Tailwater below Soffit, above Crit. Depth

Scenario

The main setup of the test model is the same as Scenario 40. The only difference is that the *1D*-channel component is being used to model the major wash. An inflow hydrograph with a maximum discharge of 5 cfs was applied to the minor wash and a uniform flow hydrograph of 530 cfs in the major wash. The 530 cfs discharge in the major wash equates to a flow depth of 1.45 feet. The intent is to test a scenario where flow exiting the storm drain at the outfall has a tailwater depth above critical depth in the pipe but below the pipe soffit elevation. The main setup of the test model consist of (1) a major wash that is 100 feet wide by 10 feet deep, (2) a parallel minor wash 20 feet wide by 8.5 feet deep and (3) the outfall invert is at the bottom of the *FLO-2D* ground surface elevation. An inflow hydrograph with a maximum discharge of 5 cfs is applied to the minor wash and a uniform discharge of 530 cfs in the major wash. The 530 cfs discharge in the major wash equates to 1.45 feet of water depth. This model was created to test if the pipe's critical depth *WSEL* or the main channel *WSEL* would be used for the boundary condition at the storm drain outfall. Using Flow Master, the pipes critical depth was identified to be 1.16 feet. A floodplain Courant value of 0.60 and *1D*-channel Courant value of 0.20 was used to stabilize the model. Plan views of the test model are shown on Figure 4.252 and Figure 4.253 and the profile view is shown on Figure 4.254.

Figure 4.252 Scenario 41: Plan View of Test Model - Elevation

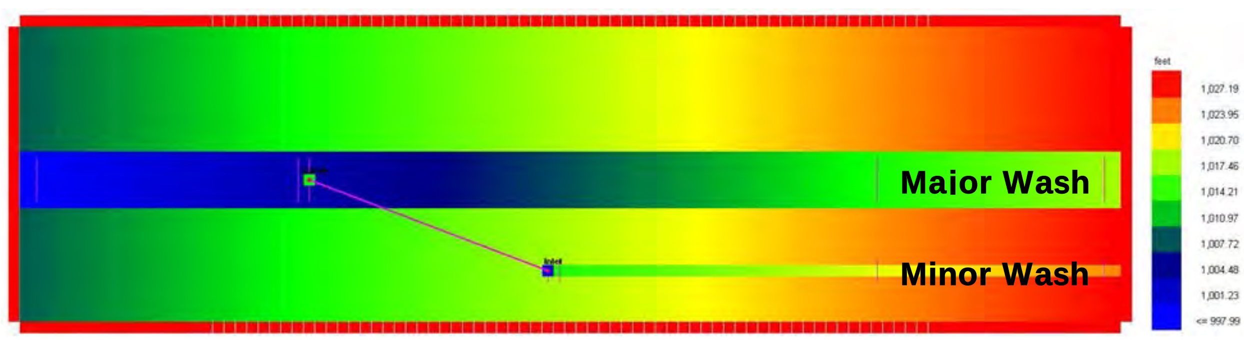
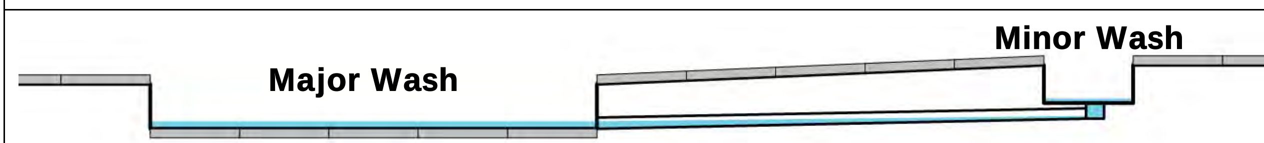


Figure 4.253 Scenario 41: Plan View of the Test Model – Max Depth

Figure 4.254 Scenario 41: Profile View of the Test Model



Benchmark

The *FLO-2D* inlet discharge and a calculated discharge is shown on Figure 4.255 to verify that the inlet is getting 5 cfs. A comparison of water surface elevations is shown on Figure 4.256.

Figure 4.255 Scenario 41: Inflow Hydrographs

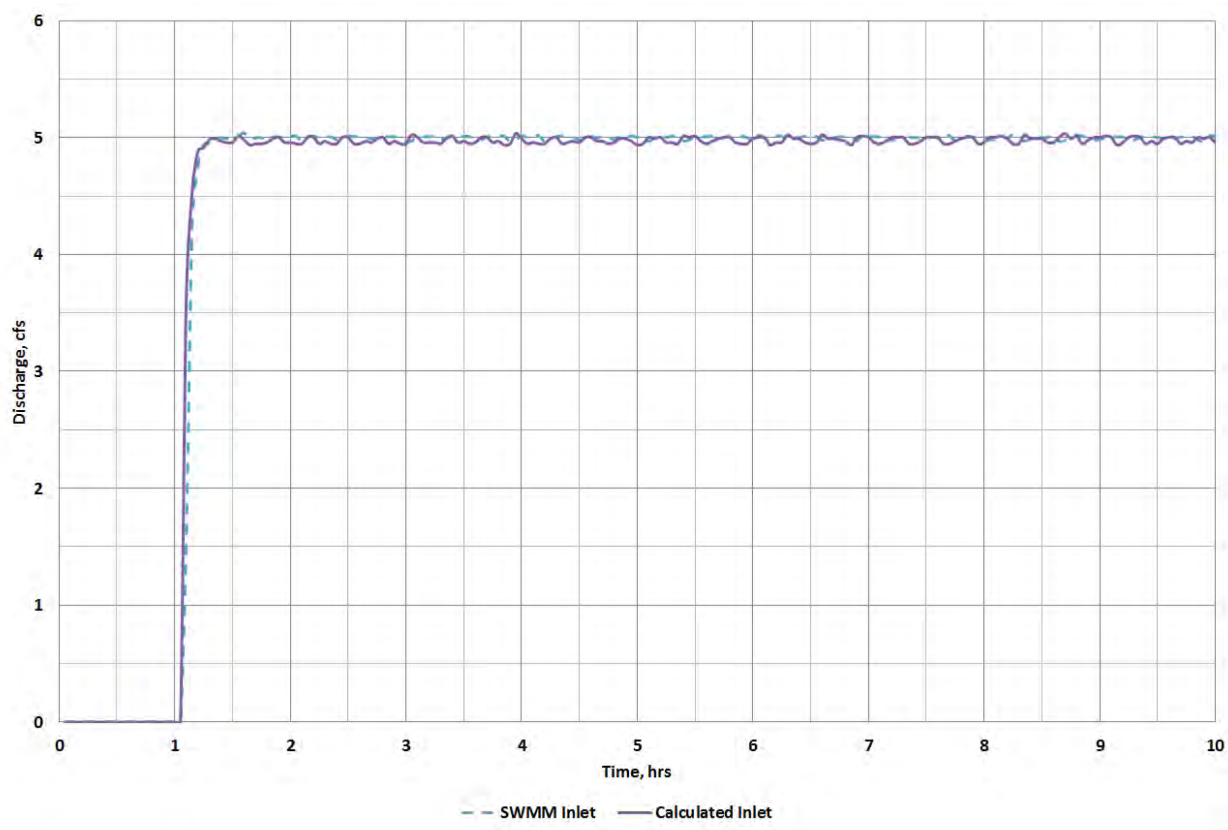
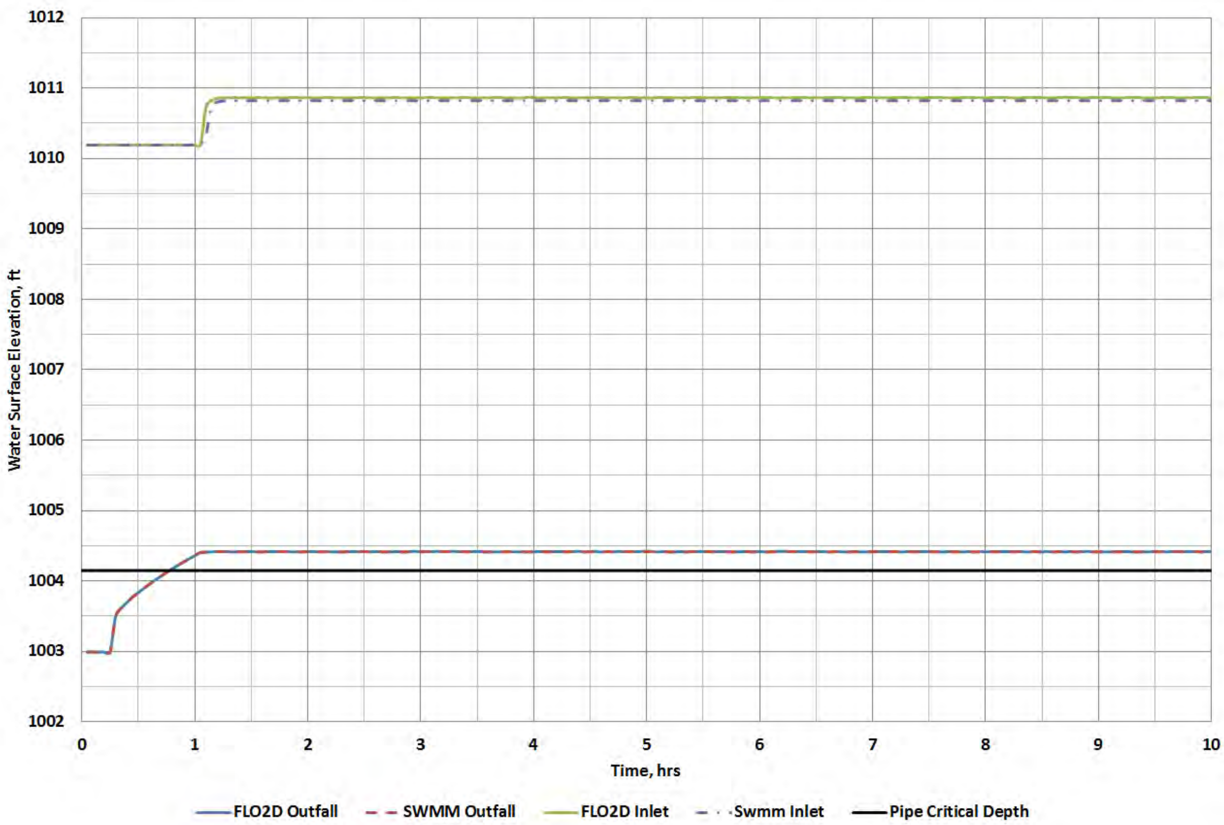


Figure 4.256 Scenario 41: Outfall & Inlet *WSEL*



Discussion

The inlet is receiving 5 cfs as shown on Figure 4.255. The *SWMM* outfall WSEL in the pipe at the outfall is the same as the *WSEL* in the major wash as shown on Figure 4.256. The *SWMM* outfall *WSEL* is higher than the pipe's critical depth *WSEL*, therefore the pipe boundary condition is based on the *WSEL* in the channel. This is acceptable for *FCDMC* purposes.

4.2.7.43 Scenario 42: Outfall to Floodplain, Tailwater above Soffit

Scenario

Test of the *FLO-2D*/*SWMM* approach for a storm drain pipe that drains from a minor wash into a major wash. The upstream end has a Type 1 Inlet: Curb Opening Inlet at Grade operating in a weir flow condition. The storm drain discharges into a main channel using an outfall. The intent is to test a scenario where flow exiting the storm drain at the outfall has a tailwater depth above the soffit of the pipe. The main setup of the test model consist of (1) a major wash that is 100 feet wide by 10 feet deep, (2) a parallel minor wash 20 feet wide by 8.5 feet deep and (3) the outfall invert is at the bottom of the *FLO-2D* ground surface elevation. An inflow hydrograph with a maximum discharge of 5 cfs was applied to the minor wash and a uniform flow hydrograph of 1,300 cfs in the major wash. The 1,300 cfs discharge in the major wash equates to a flow depth of 2.50.

This scenario was created to verify that the *WSEL* in the major wash is used as the boundary condition at the storm drain outfall. Using Flow Master, the critical depth in the pipe was calculated to be 1.16 feet. A floodplain Courant value of 0.20 was used to stabilize the model. Plan views are shown on Figure 4.257 and Figure 4.258 and a profile view on Figure 4.259.

Figure 4.257 Scenario 42: Plan View of Test Model - Elevation

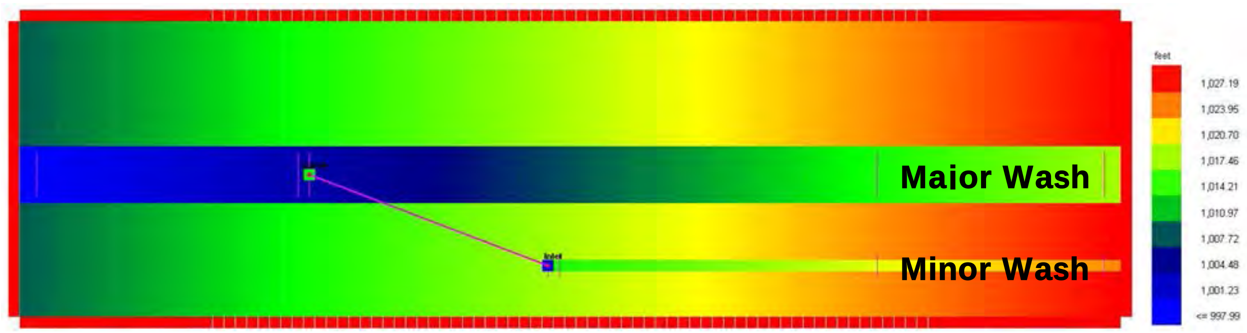


Figure 4.258 Scenario 42: Plan View of the Test Model – Max Depth

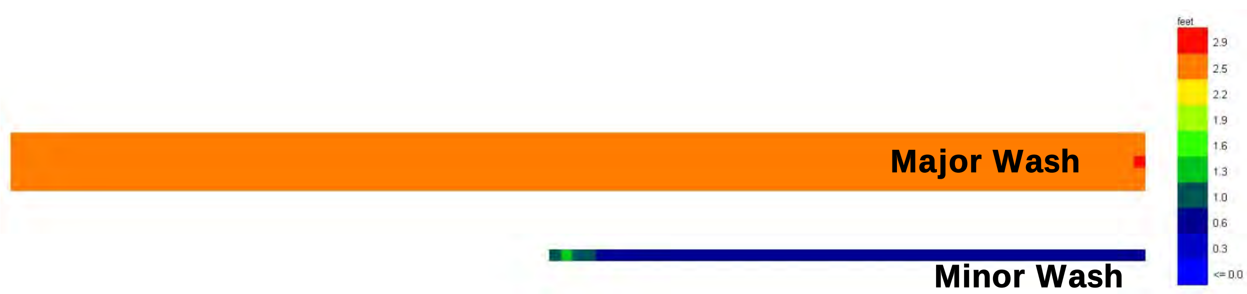
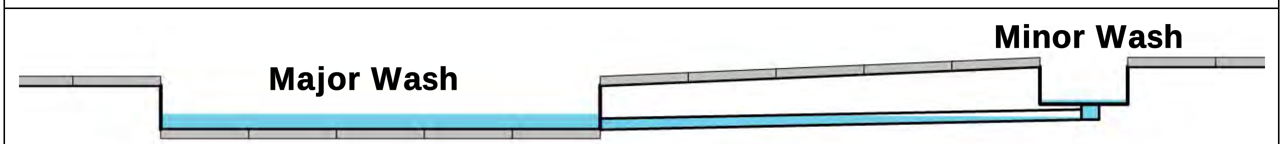


Figure 4.259 Scenario 42: Profile View of the Test Model



Benchmark

The *FLO-2D* inlet hydrograph and a hand-calculated verification hydrograph are shown on Figure 4.260 to verify that the inlet is receiving 5 cfs. A comparison of water surface elevations is shown on Figure 4.261.

Figure 4.260 Scenario 42: Inflow Hydrographs

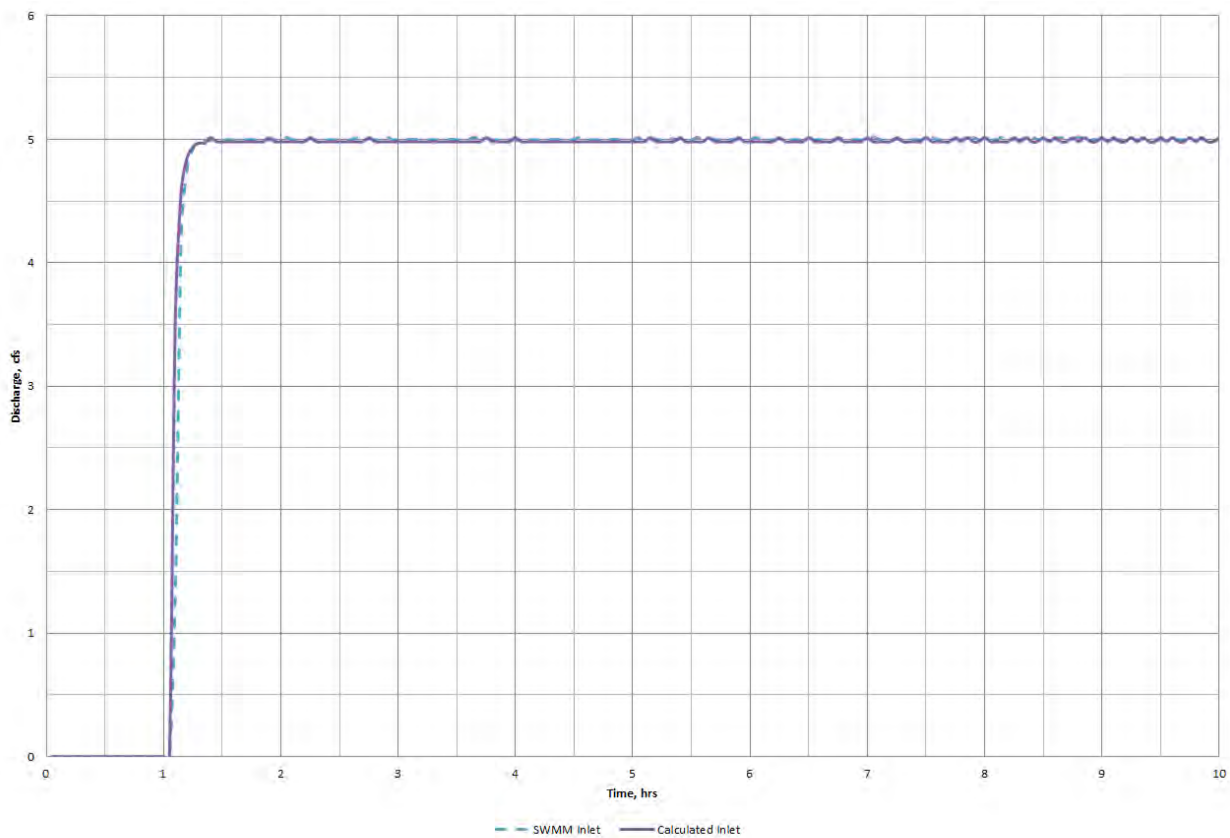
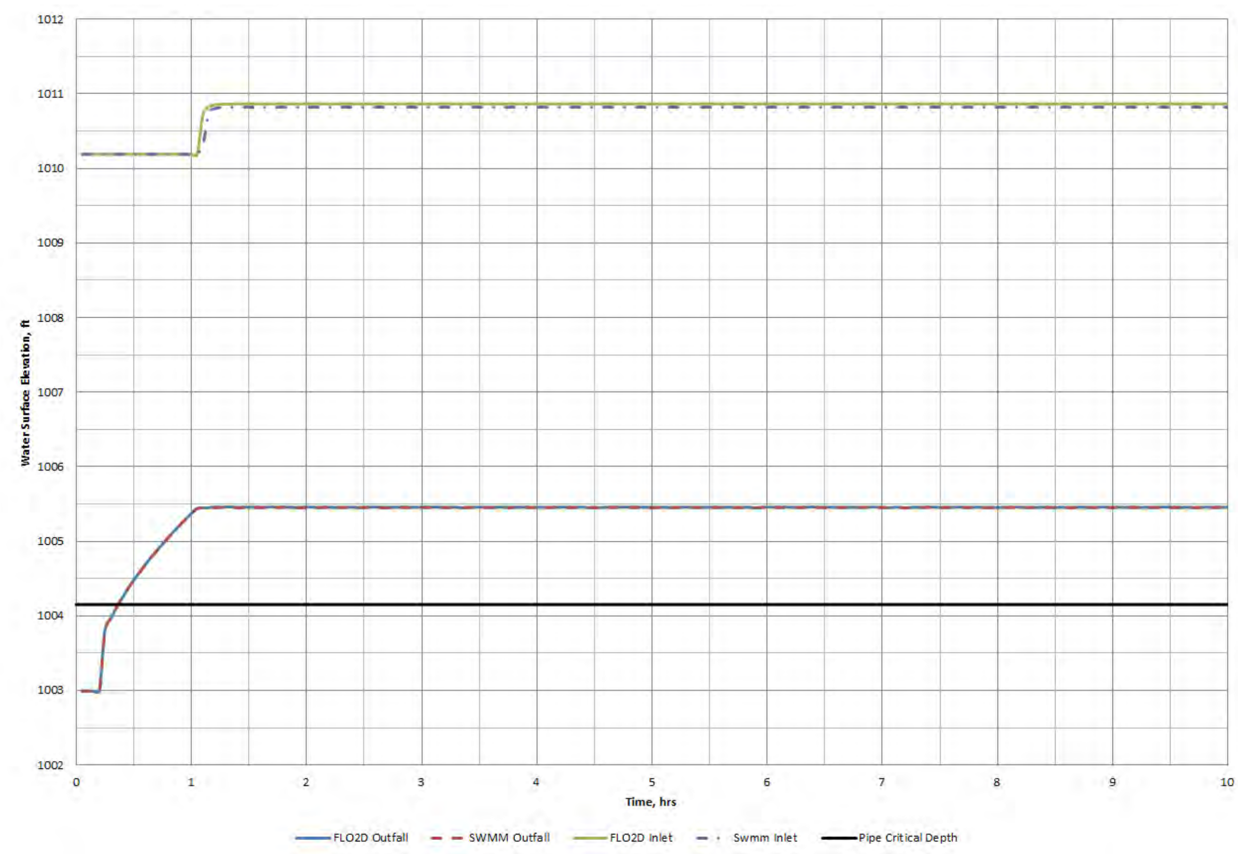


Figure 4.261 Scenario 42: Outfall & Inlet WSEL



Discussion

The inlet is receiving 5 cfs as shown on Figure 4.260. The *SWMM* outfall WSEL in the pipe is the same as the WSEL in the major wash as shown on Figure 4.261. The *SWMM* outfall *WSEL* is higher than the pipe's soffit; therefore, the pipe boundary condition is based on the *WSEL* in the channel. This is acceptable for *FCDMC* purposes.

4.2.7.44 Scenario 43: Outfall to *1D* Channel, Tailwater above Soffit

Scenario

The main setup of the test model is the same as Scenario 42. The only difference is that the *1D*-channel component is being used to model the major wash. An inflow hydrograph with a maximum discharge of 5 cfs was applied to the minor wash and a uniform discharge hydrograph of 1,300 cfs in the major wash. The 1,300 cfs discharge in the major wash equates to a flow depth of 2.50 feet. The intent is to test a scenario where flow exiting the storm drain at the outfall has a tailwater depth above the soffit of the pipe.

Using Flow Master, the critical depth in the pipe was calculated to be 1.16 feet. A floodplain Courant value of 0.20 was used to stabilize the model. Plan views are shown on Figure 4.262 and Figure 4.263 and a profile view is shown on Figure 4.264.

Figure 4.262 Scenario 43: Plan View of Test Model - Elevation

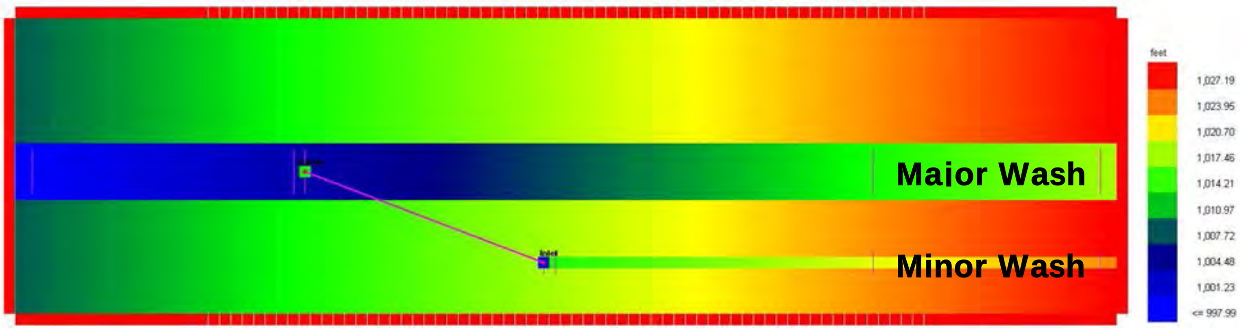
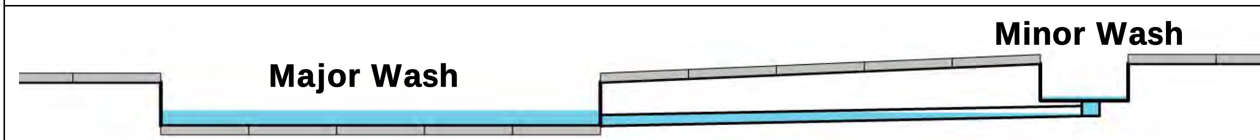


Figure 4.263 Scenario 43: Plan View of the Test Model – Max Depth

Figure 4.264 Scenario 43: Profile View of the Test Model



Benchmark

Show that the major wash water surface elevation is equal to the pipe's water surface elevation. *FLO-2D* inlet discharge and a calculated discharge is shown on Figure 4.265 to verify that the inlet is getting 5 cfs. A comparison of water surface elevations is shown on Figure 4.266.

Figure 4.265 Scenario 43: Inflow Hydrographs

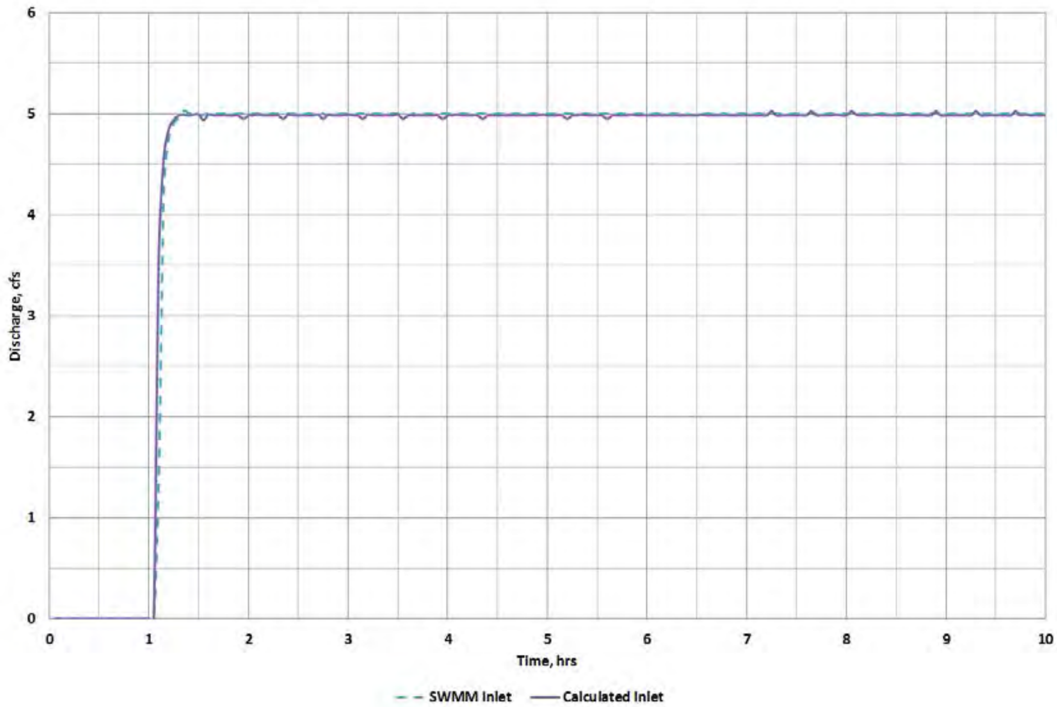
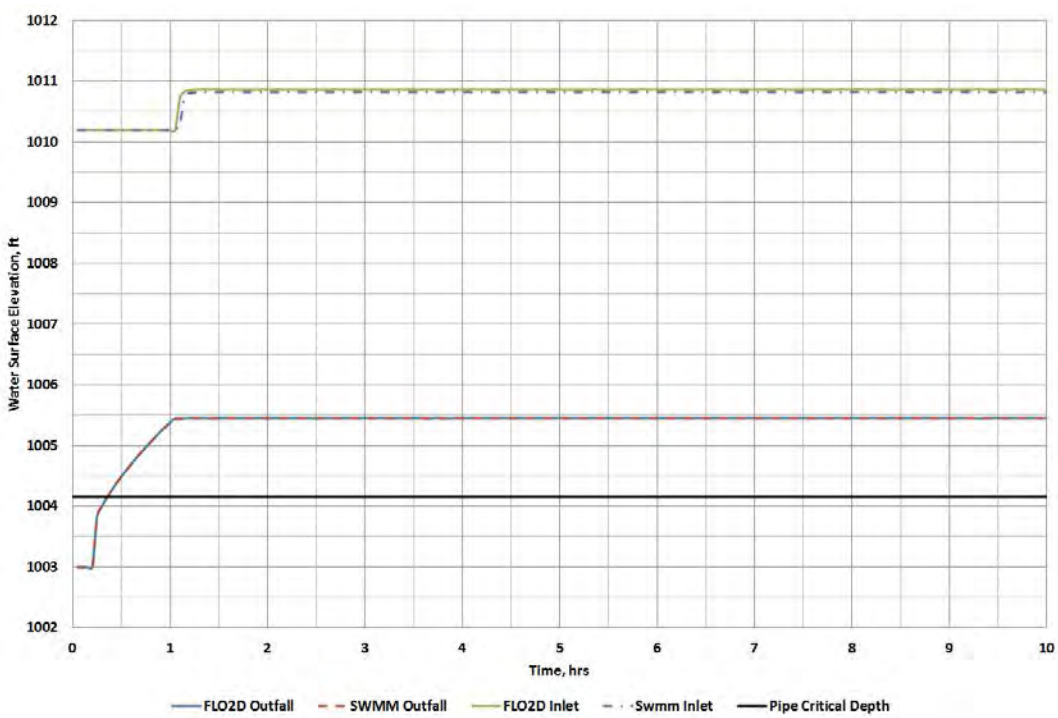


Figure 4.266 Scenario 43: Outfall & Inlet WSEL



Discussion

The inlet is receiving 5 cfs as shown on Figure 4.265. The *SWMM* outfall WSEL in the pipe at the outfall is the same as the *WSEL* in the major wash as shown on Figure 4.266. The *SWMM* outfall *WSEL* is higher than the pipe's soffit; therefore, the pipe boundary condition is based on the *WSEL* in the channel. This is acceptable for *FCDMC* purposes.

4.2.8 Sensitivity to Courant Value Assignment

There can be fairly significant differences in results from the *FLO-2D SWMM* component depending on the selection of the Courant parameter. To quantify the differences, the *SWMM* test scenarios that used a Courant number other than 0.6 were run with this setting and the results were compared to the original model. Scenarios 3, 32 and 34 were selected to provide comparisons for this report.

The Scenario 3 comparison is shown on Figure 4.267, Scenario 32 on Figure 4.268, and Scenario 34 on Figure 4.269. Note that for most scenarios a Courant of 0.6 can be used with acceptable results. Instabilities can result when using higher Courant values and both the discharge and *WSEL* results should be checked before acceptance.

Figure 4.267 Scenario 3 Courant Setting Comparison

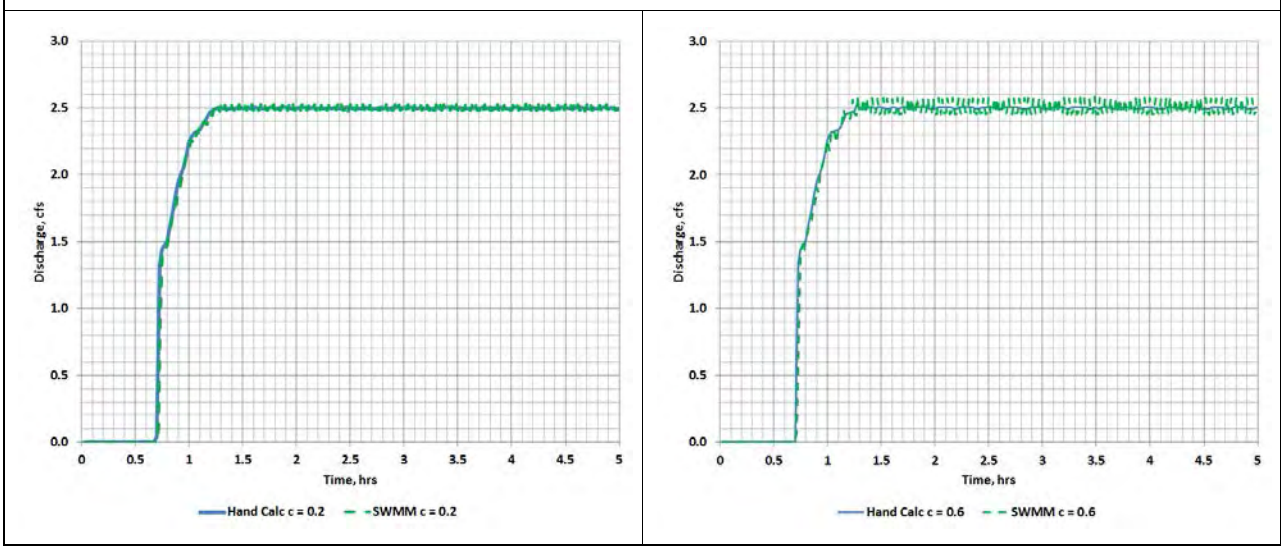


Figure 4.268 Scenario 32 Courant Setting Comparison

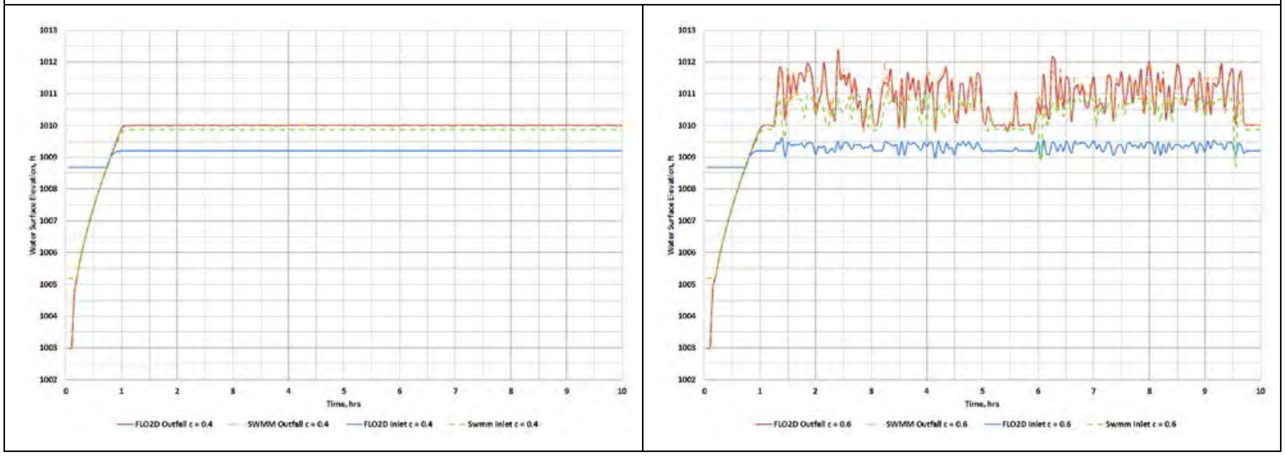
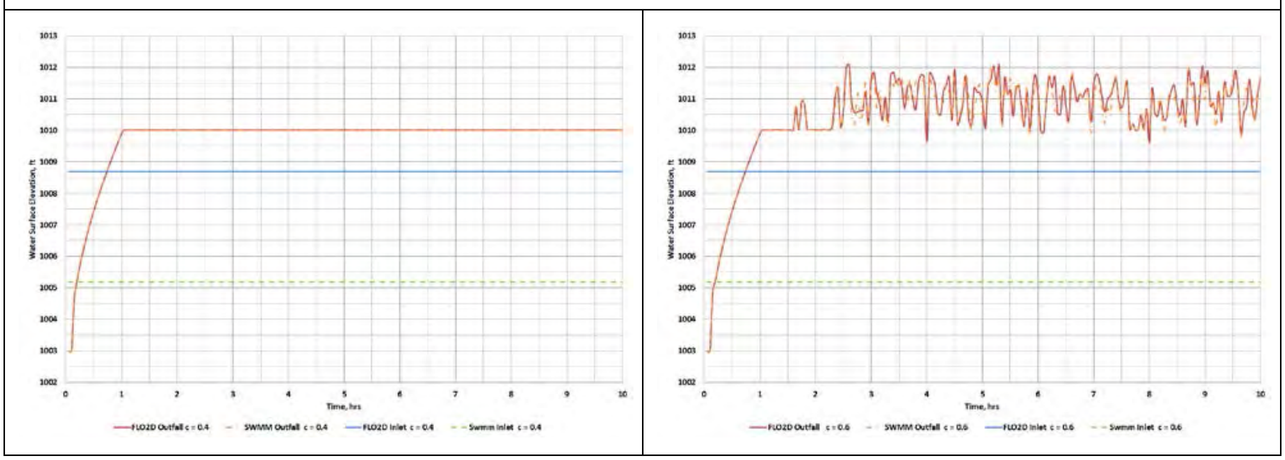


Figure 4.269 Scenario 34 Courant Setting Comparison



4.2.9 Summary and Conclusions

The *FLO-2D – SWMM* storm drain component works very well for the scenarios tested and the results check very closely with hand-calculations. The Courant parameter may need to be lowered below 0.6 in some instances for models containing *SWMM* storm drains but it is a good initial setting. A Courant of 0.4 or smaller may be necessary for final model runs. In all cases, the modeler should carefully examine the results of the *SWMM* storm drain component for instability, reasonableness and consistency and make adjustments as necessary.

Based on these test scenarios, the *FLO-2D - SWMM* component appropriately models storm drain inlet, outlet, outfall, and pipe hydraulics and is acceptable for *FCDMC* modeling purposes.

4.3 Storm Reconstitution Comparisons with Gage-Measured Hydrographs

This section documents testing of *FLO-2D* model runoff hydrographs and volumes against measured and/or observed data. These cases test the real world applicability of *FLO-2D* for performing rainfall loss and flood routing computations. Note that the *FLO-2D* capabilities for *n*-value adjustment, including depth-variable *n*, are used in these examples, which helps demonstrate their applicability. Most of the other test cases have *n*-value adjustments turned off in order to make direct comparisons with *HEC-RAS*. The approach taken for the storm reconstitution comparisons is summarized as follows:

1. Test watersheds that contain a recording stream flow gage were identified. The test watershed stream flow gages were checked for significant runoff, proper operation during the event, a current rating curve, and reasonable measured runoff hydrographs. Both the Rainbow Wash and Seven Springs Wash test watersheds fit this case.
2. Test watersheds that contain a stage recording gage were identified. This is the case for the Guadalupe Flood Retarding Structure (*FRS*). The stage recording gage was checked for proper operation during the storm event, a current rating curve, and reasonable measured stage hydrograph. This gage provided an accurate measure of runoff volume over time stored in the Guadalupe *FRS*.
3. Test watersheds without a stream flow gage were identified. For the Hohokam *ADMS*, a stream flow gage did not exist. Physical evidence of flood depths and discharges were collected for the study area. A wall failure and resulting redirected flood flows was documented.
4. Storms of interest over test watersheds were identified. METSTAT, Inc., teamed with Weather Decision Technologies, Inc., was used under contract to post-process the NEXRAD data for each storm. Physical and atmospheric abnormities were filtered out, multiple radar data sets were merged if appropriate, and the resulting data adjusted using measured rainfall at physical rain gage locations (Gage Adjusted Radar Rainfall, *GARR*). The data (rainfall depth during each time period) was provided in an *ESRI* ASCII grid format at a spatial resolution of 1 square kilometer and a temporal resolution of 5 minutes. A report documenting each storm was also provided. The test watersheds identified include:
   A. Rainbow Wash at SR 85.
   B. Seven Springs Wash at Seven Springs Road and National Forest Road 254.
   C. Guadalupe Flood Retarding Structure (*FRS*).
   D. Hohokam *ADMS* Study Area.
   E. Refer to Figure 4.270 for a map showing the locations of the four test case areas.
5. Rainfall loss parameters were field measured. Rainfall loss parameters were field measured for the Rainbow Wash and Seven Springs Wash test watersheds. This work was done under an

Intergovernmental Agreement (*IGA*) with the Desert Research Institute (*DRI*), University of Nevada Las Vegas. Measurement methods consisted of a rainfall simulator, tension infiltrometers, and air permeameters. The measured data was used to estimate *IA* and *G&A* parameters for each test site. A geomorphic analysis was prepared for the test watersheds. Geomorphic map unit polygons in *GIS* feature class format were prepared for the entire watershed of both study areas. Representative locations within each geomorphic unit were selected for performing the rainfall loss measurements. Individual reports were prepared by *DRI* for both test watersheds and published in the literature.

6. Rainfall loss parameters for the Guadalupe *FRS* test watershed were not available. Representative soil samples were taken and soil tests performed to obtain the percentage of clay, silt, sand, gravel and organic matter for each sample. This data was used to estimate the soil texture classification and assign *G&A* parameters based on the *DDM Hydrology*. There is extensive rock outcrop within the watershed. The areas of rock outcrop were estimated by field survey and reconnaissance and using available aerial photographs.
7. Surface Feature Characterizations were developed. Polygons of various surface features were prepared for each study area. Feature types included asphalt, concrete, buildings, canopies, natural impervious areas, various vegetation categories, wash bottoms, etc. Walls were defined using a separate polyline Feature Class. These features were used as the basis for assigning *IA*, *RTIMP*, *n*-value, and flow obstructions to the *FLO-2D* models.
8. A *FLO-2D* model was built of each test watershed using the information from steps 1-7 and available topographic surface models and the results analyzed and compared with the gage measured hydrographs. Only minor calibration adjustments were done.
9. Available *HEC-1* models for the test watersheds were modified to apply each storm's rainfall and the results compared with the gage measured hydrographs and the *FLO-2D* results.

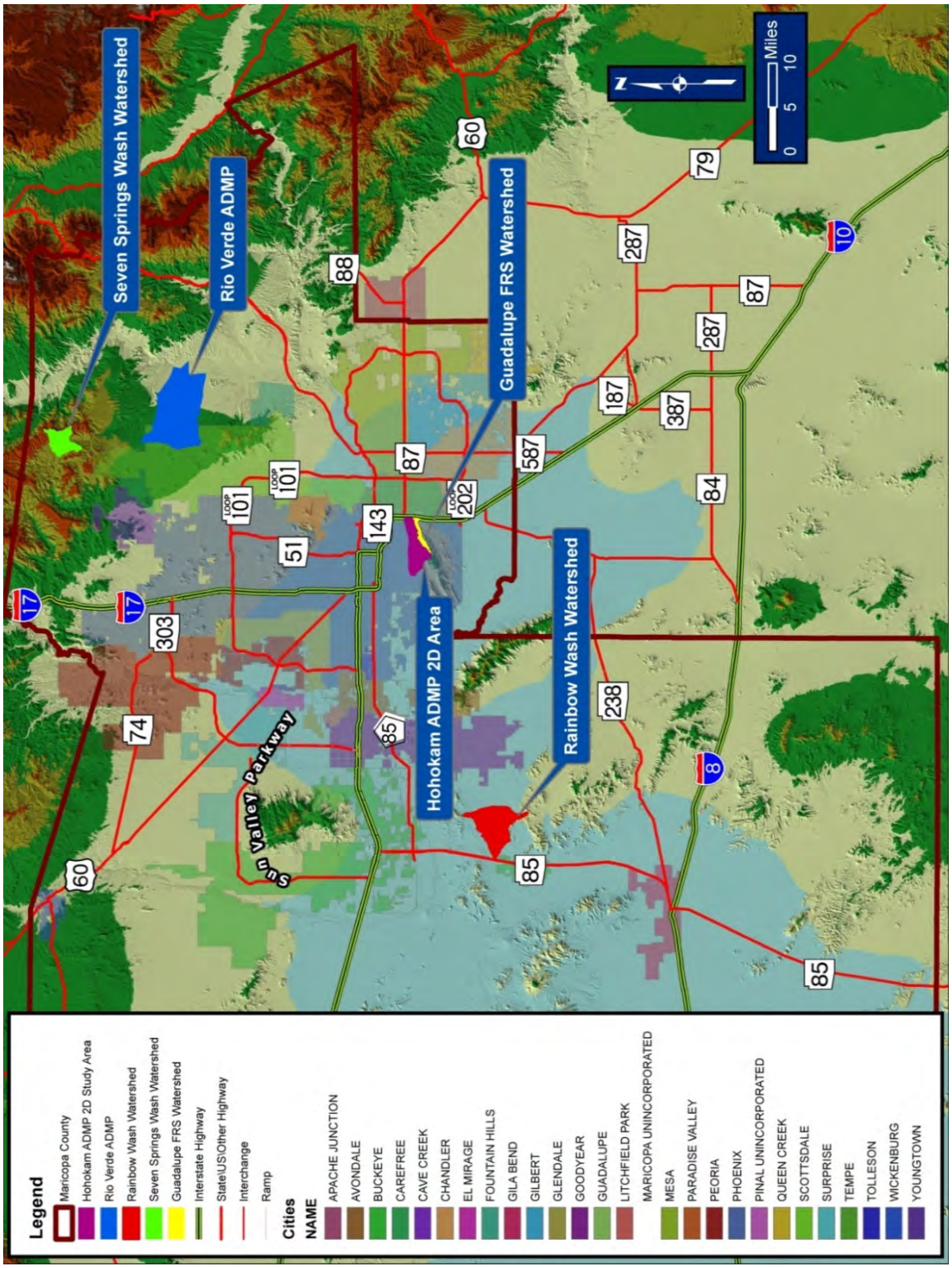
There are fairly significant potential accuracy issues when performing storm reconstitution. These accuracy issues apply to all models, including *FLO-2D*. The possible inaccuracies include, but are not limited to, the following:

1. The resolution and inherent inaccuracies of the *NEXRAD* data.
2. Potential inaccuracies in rain gage and stream flow gage measurements.
3. The resolution of the *FLO-2D* model.
4. The rainfall loss estimation method.
5. The accuracy of the topographic mapping.
6. The estimated surface roughness values selected for use in *FLO-2D.*
7. The estimated surface roughness values selected for use in the stream flow gage hydraulic rating curve models.

8. Channel bed and bank movement during the event.

Even factoring in the potential inaccuracies, the storm reconstitution models provide meaningful information for *FLO-2D* model verification. The following sections document the findings of the verification effort based on storm reconstitutions.

Figure 4.270 Actual Storm Test Cases Location Map



4.3.1.2 Rainbow Wash Test Cases

General

The Rainbow Wash in Maricopa County was selected for a storm reconstitution test case. The location of the Rainbow Wash watershed is shown on Figure 4.14 and on Figure 4.270. The watershed for Rainbow Wash at SR 85 is shown on Figure 4.15 and a geomorphic characterization on Figure 4.272. The watershed area is approximately 17.99 square miles. *FCDMC* owns and maintains a recording pressure tranducer stream flow gage (*FCDMC* Gage 6953) and a rainfall gage (*FCDMC* Gage 6950) just upstream of the SR 85 crossing, which has been in operation since November 11, 2000. The wash is a well-defined natural channel with a dominately trapezoidal cross section, a sand bed underlain by caliche, and well vegetated banks. Refer to photographs of the wash at the gage site shown on Figure 4.271.

Several major flows have been recorded at this gage site in recent years. Four (4) storms were selected as test cases and are listed in Table 4.44.

Figure 4.271 FCDMC Rainbow Wash Gage Site Photographs

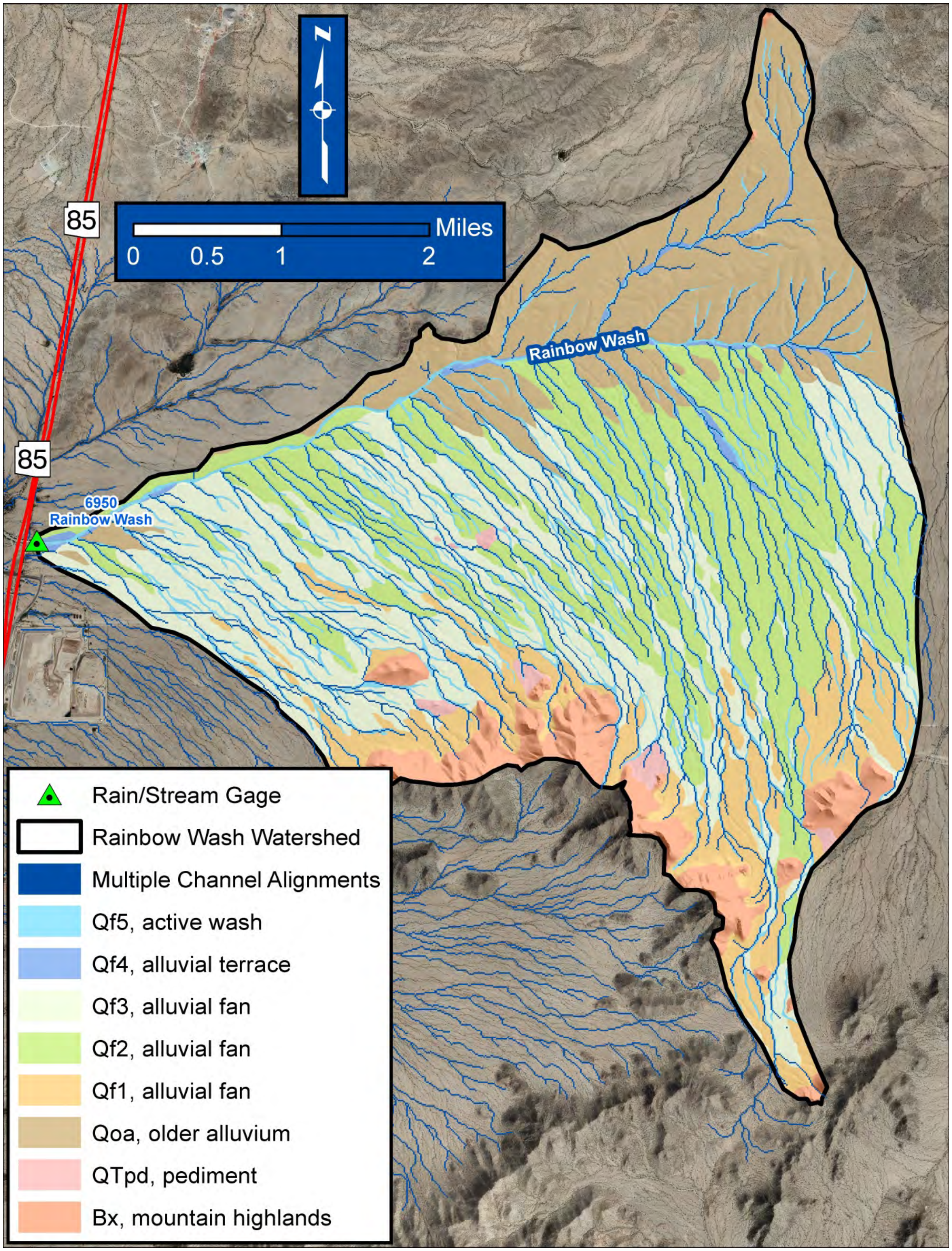
Looking at standpipe and sensor location

Looking downstream from gage sensor

Table 4.44 Rainbow Wash Test Case Storms

Storm Date	Maximum Gage Height	Estimated Peak Discharge, cfs
(1)	(2)	(3)
August 7-8, 2008	3.24	732
January 18-23, 2010	2.33	570
July 29, 2010	3.64	1,625
August 21, 2012	5.00	>3,200 (rating exceeded)

Figure 4.272 Rainbow Wash Geomorphic Units



The *FLO-2D* model used to simulate these storms was developed as a part of the Gillespie *ADMP* project. The model development and parameters are documented in *Gillespie Area Drainage Master Study Hydrology* (Stantec Consult.Services, Inc., 2013). The key information for the model of the Rainbow Wash Watershed includes:

- FLO-2D Pro Build 14.11.09
- 50-foot grid size. The ADMP model is intended for hydrology, not detailed hydraulics.
- *G&A* rainfall loss method using field measured parameters.
- *FLO-2D* multiple channels option applied to supplement the relatively large grid size and to simulate a number of small parallel natural channels that convey most of the runoff from upstream areas into the Rainbow Wash. The dark blue multiple channel alignment lines shown on Figure 4.272 are where the multiple channel option was applied.

Test locations for field measured *G&A* parameters were sited within the geomorphic units shown on Figure 4.272. The geomorphic assessment, field testing, and generation of parameters is documented in *Hydraulic Characteristics of Soil in Rainbow Wash Watershed in Maricopa County* (Chen, et al., 2010).

Storm *GARR* data was generated by METSTAT, Inc. for each storm as briefly described previously. Refer to the following reports for documentation of the generation of the GARR data:

1. *Storm Precipitation Re-Analysis Report, Storm of August 9, 2005, Maricopa County, Arizona (Rainbow Wash Watershed), SPAS Storm #1300* (METSTAT, Inc., 2013b)
2. *Storm Precipitation Re-Analysis Report, Storm of August 7-8, 2008, Maricopa County, Arizona (Rainbow Wash Watershed), SPAS Storm #1204* (METSTAT, Inc., 2012a).
3. *Storm Precipitation Analysis Report, Storm of January 18-23, 2010, Maricopa County, Arizona SPAS Storm #1238* (METSTAT, Inc., 2012b).
4. *Storm Precipitation Analysis Report, Storm of July 29, 2010, Maricopa County, Arizona SPAS Storm #1196* (METSTAT, Inc., 2012c).
5. *Storm Precipitation Analysis Report, Storm of August 21, 2012, Maricopa County, Arizona SPAS Storm #1287* (METSTAT, Inc., 2013a).

The *GARR* data was used to create the *FLO-2D* RAINCELL.DAT input data file, which simulates the actual moving storm rainfall using 5-minute time intervals. The resulting runoff hydrograph for each storm was then compared with the measured hydrograph from *FCDMC* Gage 6953.

The *HEC-1* model from the *FEMA* Flood Insurance Study (FIS) for Rainbow Wash was also used to model each storm. The rainfall loss parameters were left unchanged and not revised to use the field-measured G&A parameters. The 5-minute rainfall data was averaged within each model sub-basin to create an individual rainfall distribution for every sub-basin. Refer to *Gila Bend Canal, Floodplain Delineation Study, Gillespie Dam to Gila Bend, FCD 90-06, Hydrology Report* (Donahue & Associates, Inc., 1991) for the supporting information for the FIS HEC-1 model. The results are presented and discussed in the following sections. Note that the time base of the *FLO-2D* and *HEC-1* models were shifted to better align the model results with the measured for comparison purposes.

Storm of August 9, 2005

The hydrograph results for this storm are shown on Figure 4.273. The total *GARR* rainfall for the storm and the rainfall distribution measured at *FCDMC* Gage 6950 are shown on Figure 4.274. This storm was a short-duration summer storm with a total duration in the area of Gage 6950 of about 1 hour and a maximum rainfall amount on the watershed of about 1.93 inches. Note that the rainfall was not uniform over most of the watershed, with the most rain falling near SR 85.

The *FLO-2D* model results are lower than the measured. The *FLO-2D* peak discharge is about half of the measured, but the general shape is similar. *FLO-2D* is producing less runoff volume than measured for this case. The *HEC-1* model does not match at all with the measured. The averaging of the rainfall within each sub-basin results in the loss of high intensity rainfall over smaller areas. The *FLO-2D* model does a better job of representing the high intensity rainfall areas.

A pre-flood aerial photograph (Dec 2004) of Rainbow Wash at SR 85 is shown on Figure 4.275. The estimated flood limits based on a post-flood field reconnaissance and post-flood aerial photographs, are also shown. These limits represent the boundary of the high-discharge flow areas. The visible effects of high discharge flow areas are easier to define from the aerial photographs than shallow flow areas with lower velocity. The same flood limits are shown with the post-flood aerial photographs (Jan 2006) on Figure 4.276. The *FLO-2D* model maximum flood depths are added to the post-flood map on Figure 4.277. The *FLO-2D* maximum depth results compare well with the estimated actual flood limits, although it is apparent that the *FLO-2D* results are lower than actual.

For this case, *FLO-2D* provides a reasonable representation of the storm event within the possible range of error, and a much better representation than the *HEC-1* model. The possible errors for the measured hydrograph include measurement error associated with the stream flow gage, and the numerous possible errors associated with the hydraulic model the gage rating curve is based on. The possible *FLO-2D* and *HEC-1* models errors include a plus or minus 10% to 20% range for the *GARR* data, as well as the normal errors for models of these types.

Figure 4.273 Hydrograph Results for Storm of August 9, 2005

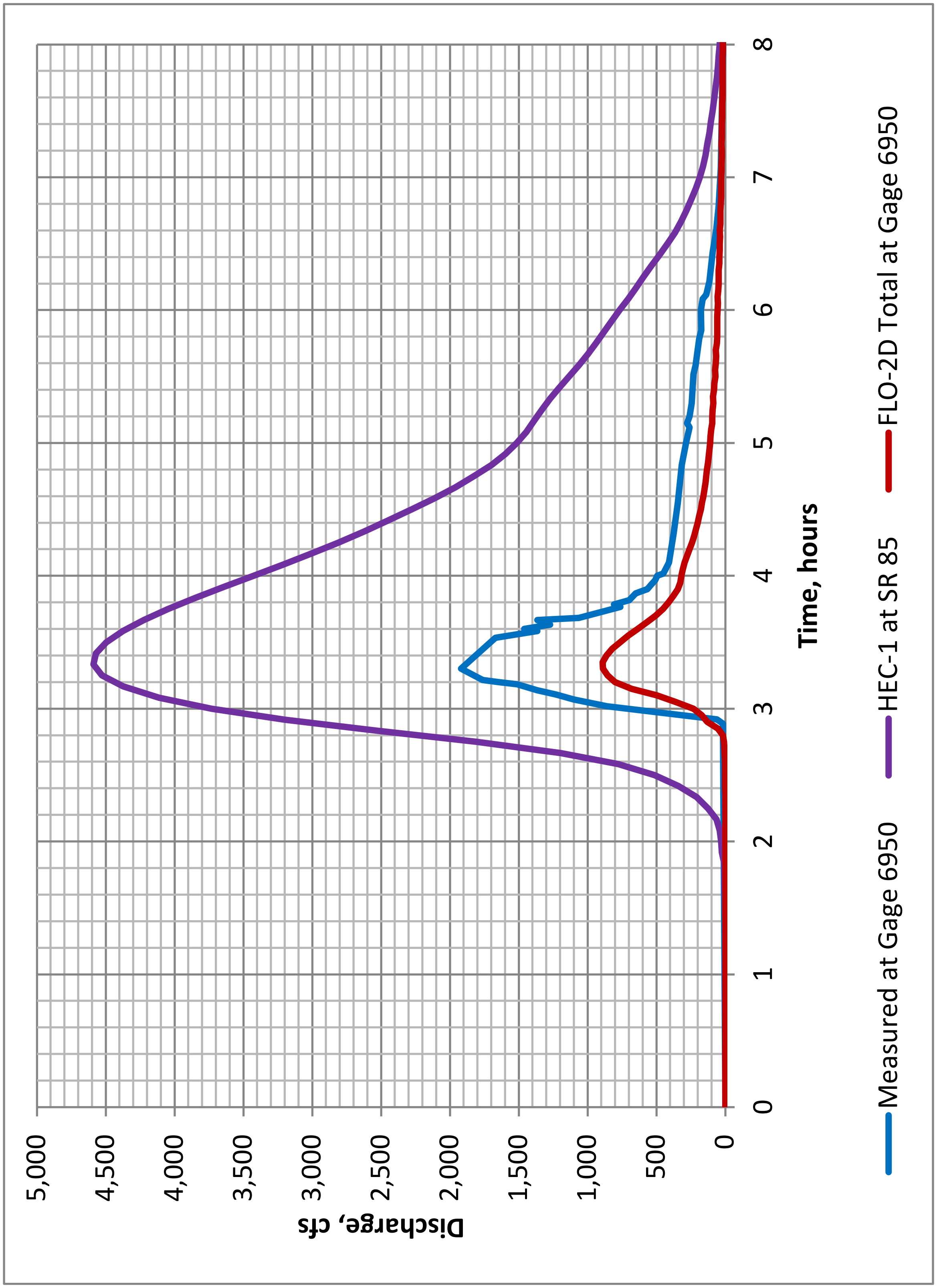
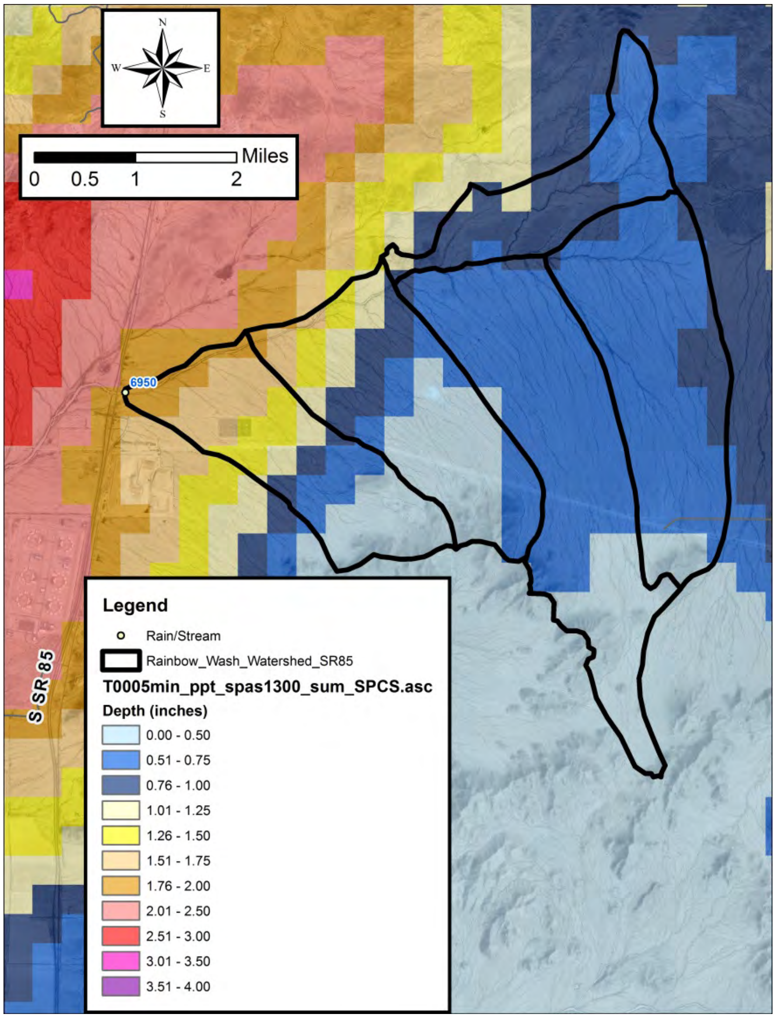


Figure 4.274 Total *GARR* Rainfall for Storm of August 9, 2005



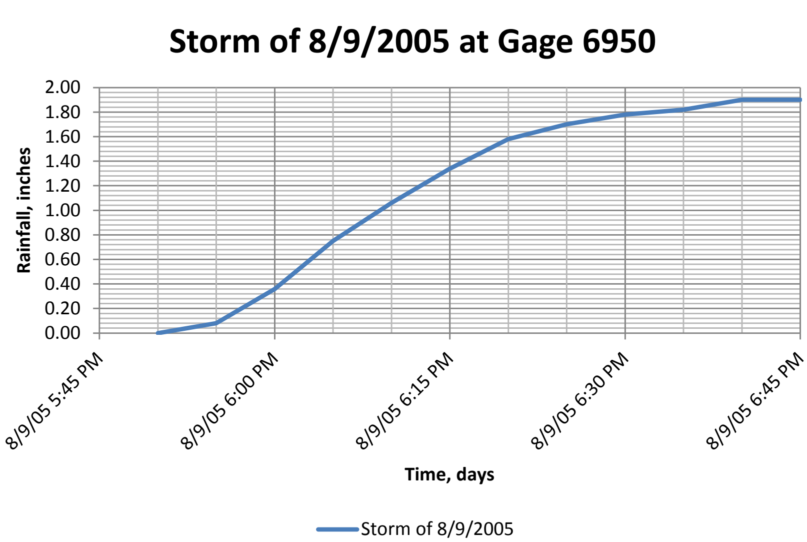


Figure 4.275 April 2005 Aerial Photograph – Pre-Flood

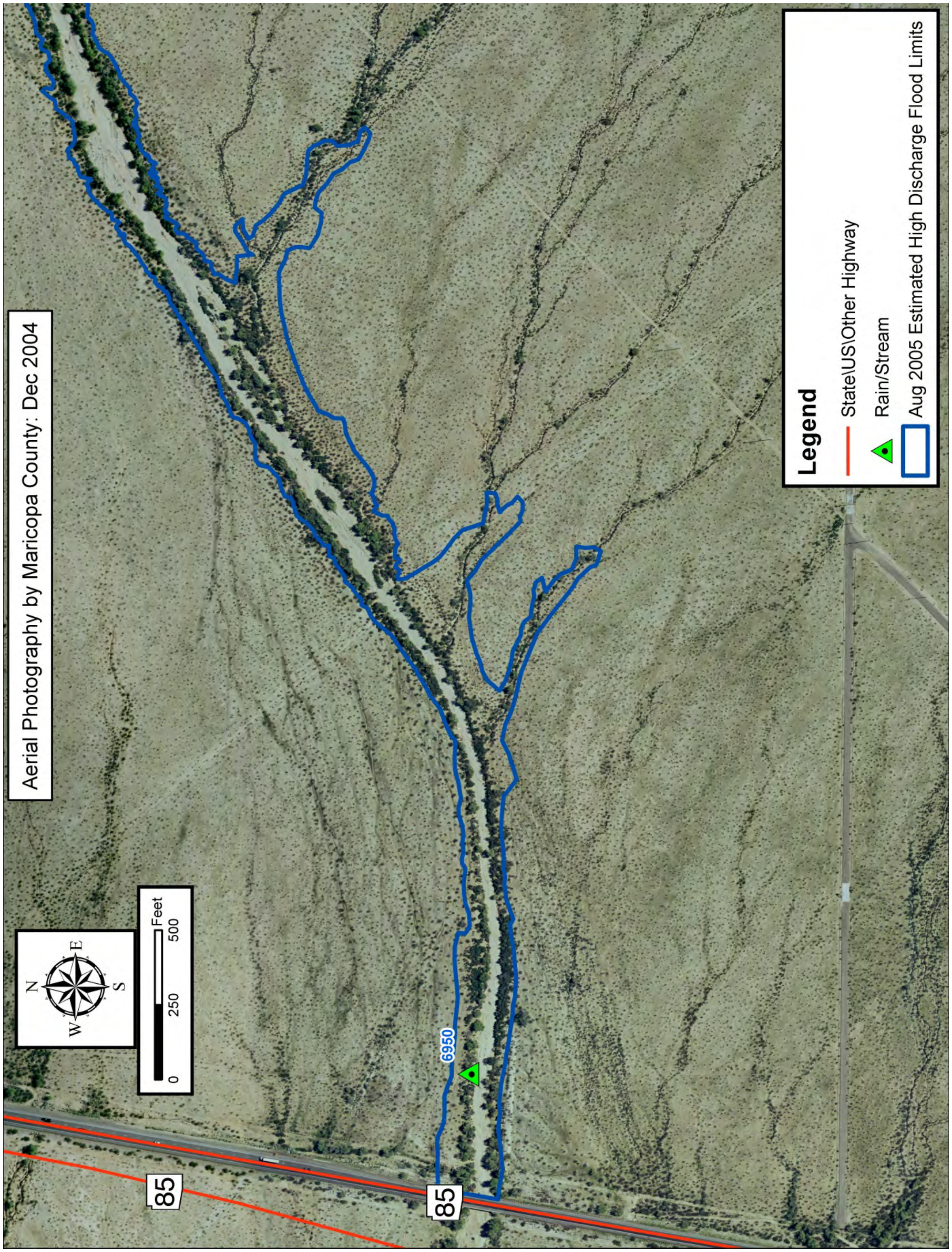


Figure 4.276 January 2006 Aerial Photograph – Post-Flood

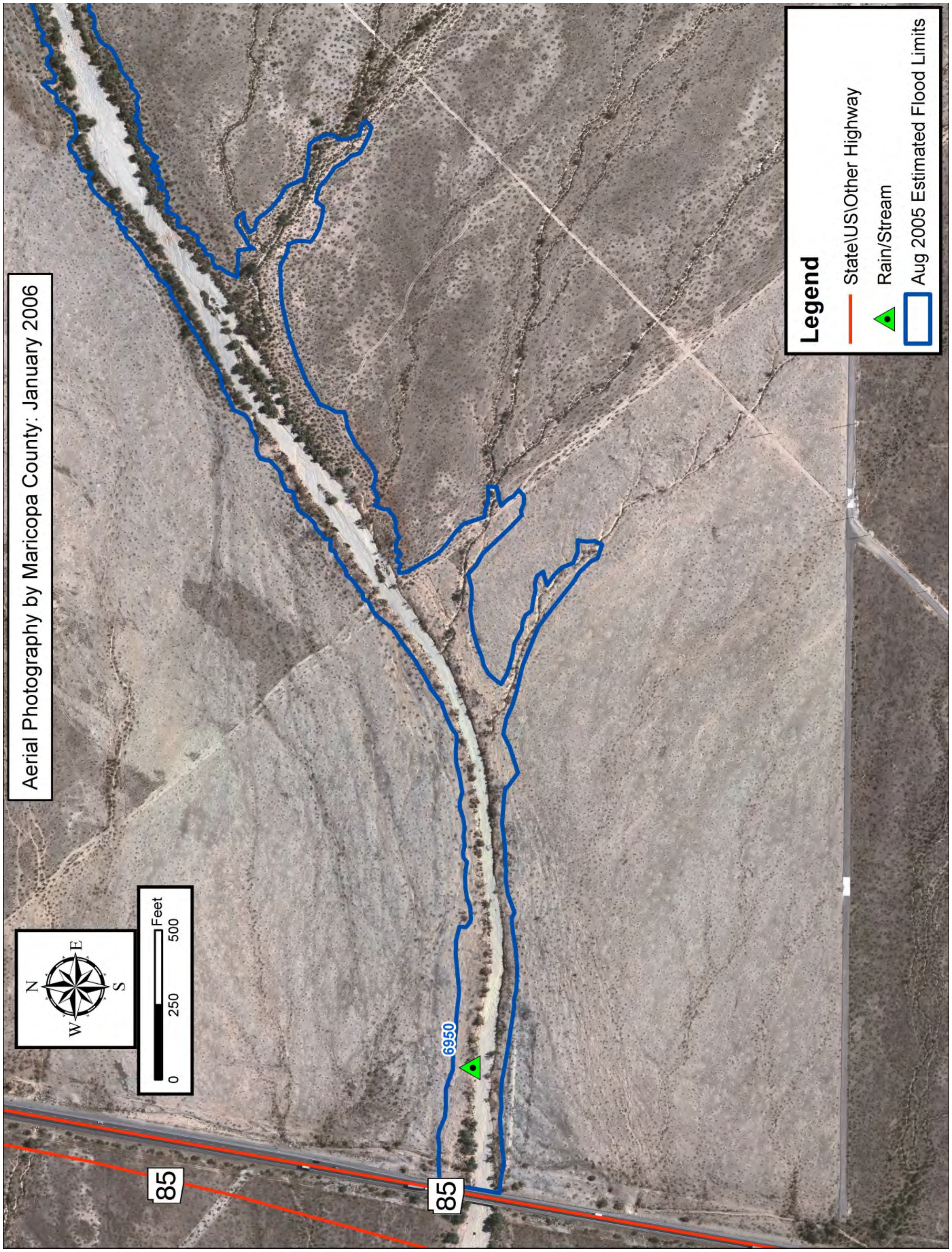
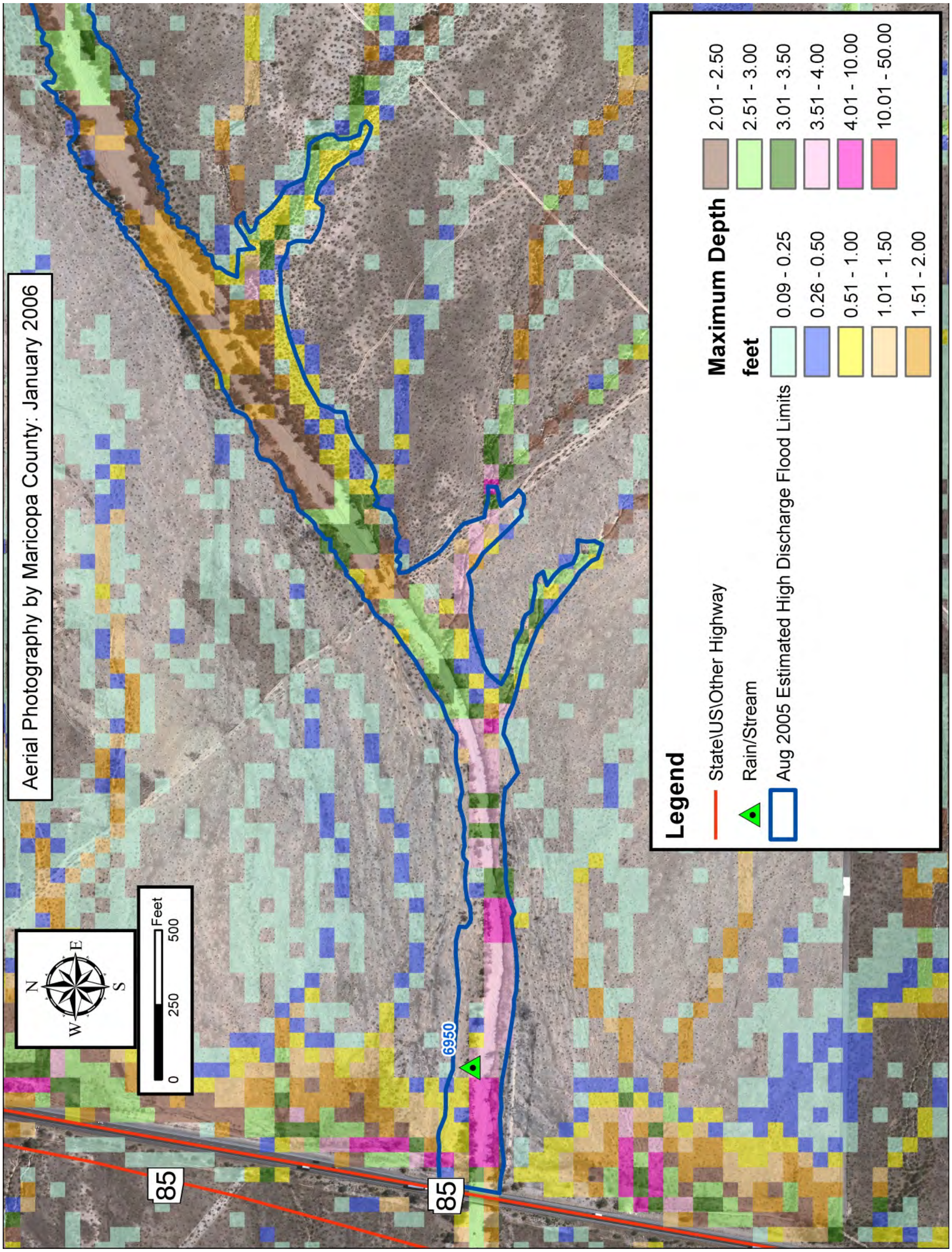


Figure 4.277 *FLO-2D* Maximum Depth Results



Storm of August 7-8, 2008

The hydrograph results for this storm are shown on Figure 4.278. The total *GARR* rainfall for the storm and the rainfall distribution measured at *FCDMC* Gage 6950 are shown on Figure 4.279. This storm was a long-duration summer storm with a total duration in the area of about 12 hours and a maximum rainfall amount on the watershed of about 2.08 inches. The actual rainfall duration over this watershed was about 4 hours. Note that the rainfall was fairly uniform over most of the watershed, with less rain near SR 85.

Also note that the *FLO-2D* model results are a fairly close match to the measured. The *FLO-2D* hydrograph rising limb is missing influx of runoff and the peak discharge is higher than measured, but the general shape and runoff magnitudes are similar. *FLO-2D* is producing close to the same runoff volume as measured for this case. The *HEC-1* model does not match at all with the measured. The averaging of the rainfall within each sub-basin results in the loss of high intensity rainfall over smaller areas. The *FLO-2D* model does a better job of representing the high intensity rainfall areas. For this case, *FLO-2D* provides a reasonable representation of the storm event within the possible range of error, and a much better representation than the *HEC-1* model. The possible errors for the measured hydrograph include measurement error associated with the stream flow gage, and the numerous possible errors associated with the hydraulic model the gage rating curve is based on. The possible *FLO-2D* and *HEC-1* models errors include a plus or minus 10% to 20% range for the *GARR* data, as well as the normal errors for models of these types.

Figure 4.278 Hydrograph Results for Storm of August 7-8, 2008

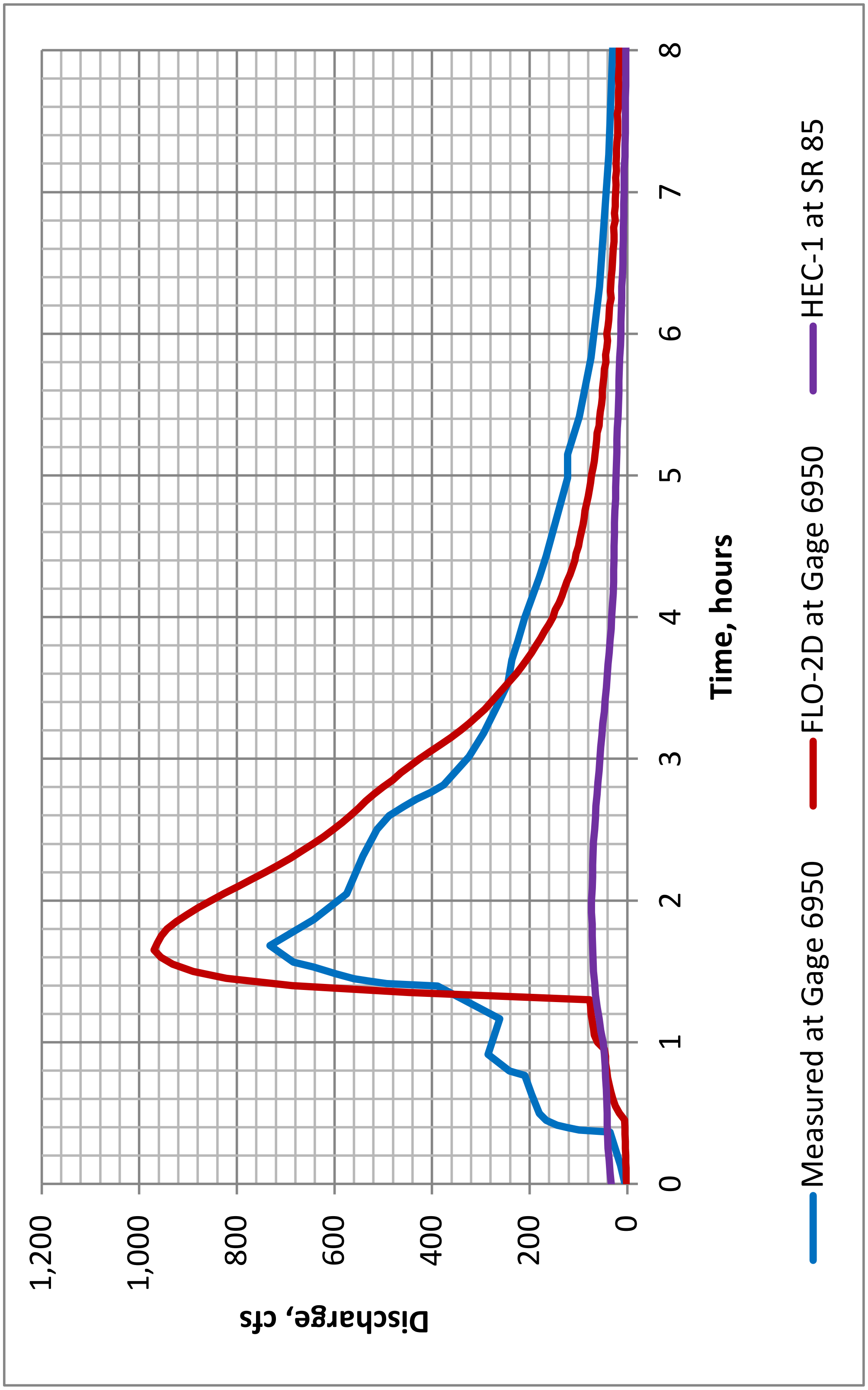
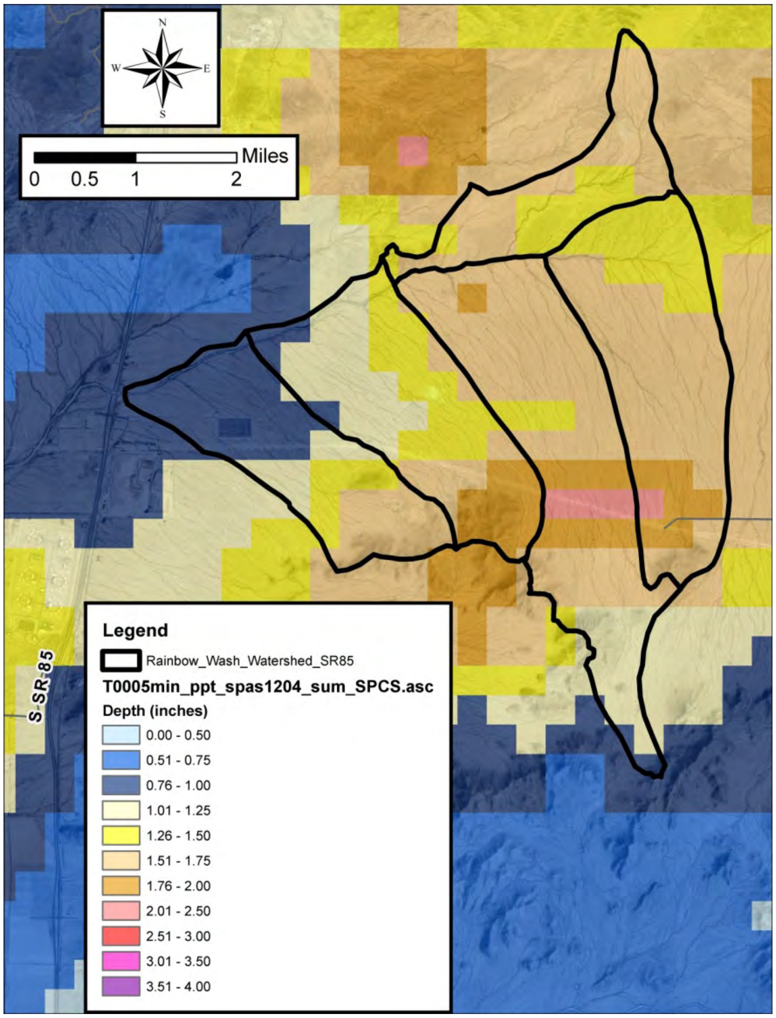
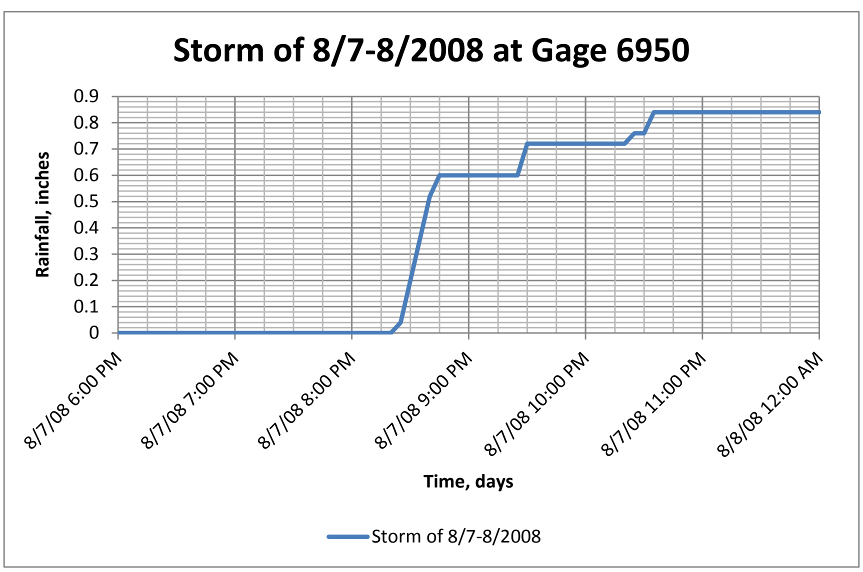


Figure 4.279 Total *GARR* Rainfall for Storm of August 7-8, 2008





<u>Storm of January 18-23, 2010</u>

The hydrograph results for this storm are shown on Figure 4.280. The total *GARR* rainfall for the storm and the rainfall distribution measured at *FCDMC* Gage 6950 are shown on Figure 4.281. This storm was a long-duration winter storm with a total duration in the area of about 95 hours and a maximum rainfall amount on the watershed of about 3.03 inches. The actual rainfall duration over this watershed was about 2.5 days with a 2 day gap between the first inch and the next 1.6 inches. The first inch resulted in very little runoff, while the second 1.5 to 2.0 inches of rain on a saturated watershed resulted in more significant runoff. Note that the rainfall was very uniform over most of the watershed.

The *FLO-2D* model hydrograph is similar but has a slightly lower peak discharge than the measured. The rising limb of the *FLO-2D* model hydrograph shows an initial influx of runoff that was not picked up by the gage. The total runoff volume measured at the gage was 87 ac-ft. The runoff volume from the *FLO-2D* model was 179 ac-ft, over twice what was measured at the gage. The gage hydrograph has a very sharp spiked high peak discharge that is not represented in the *FLO-2D* model hydrograph. The spike in the gage hydrograph may be due to wave action. A 6-inch increase in depth at that stage results in a large increase in discharge. The *HEC-1* model hydrograph is a better match than was seen for the 2008 storm, but still much lower than measured. For this case, *FLO-2D* provides a reasonable representation of the storm peak discharge and hydrograph shape, and a better representation than the *HEC-1* model.

Figure 4.280 Hydrograph Results for Storm of January 18-23, 2010

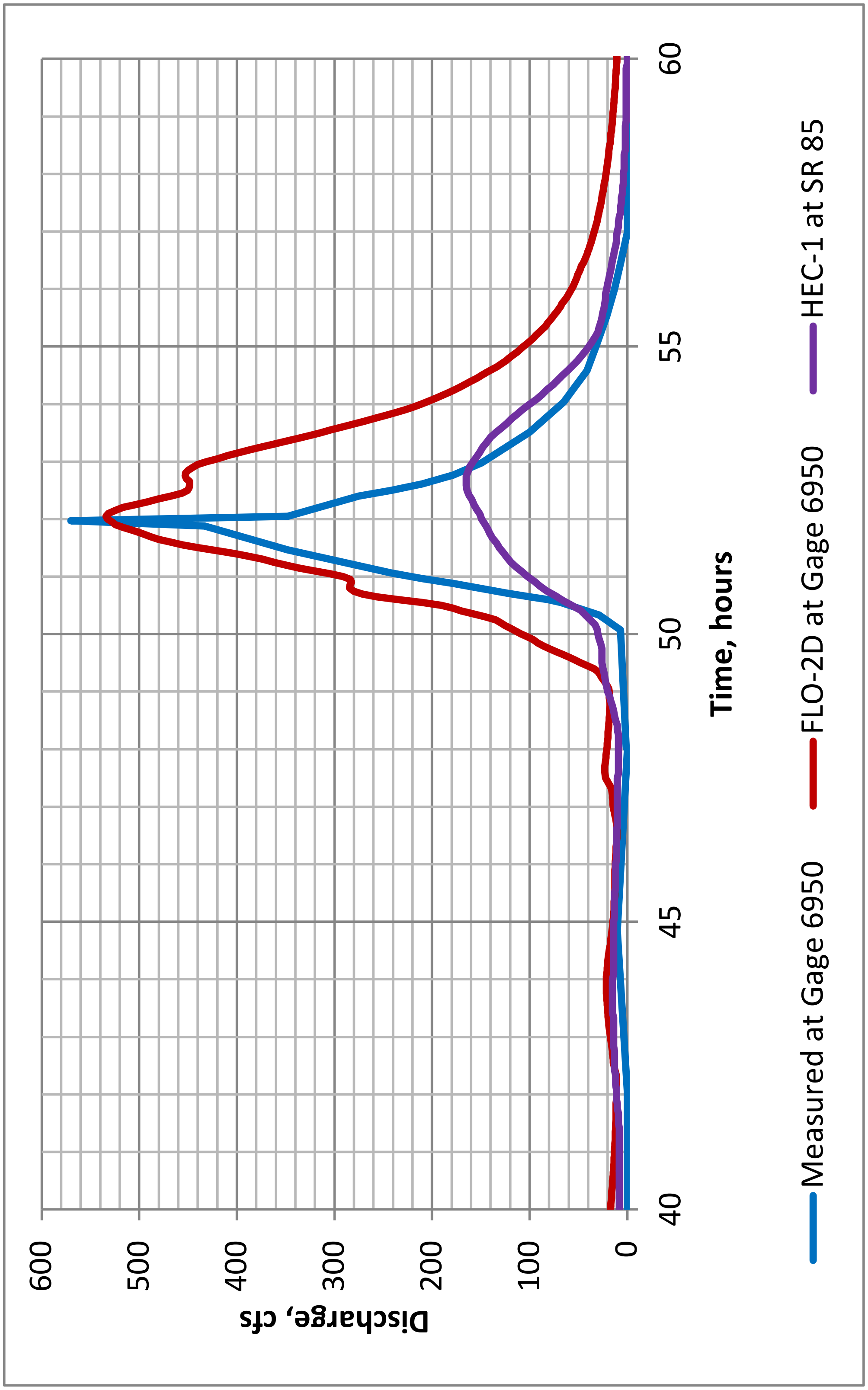
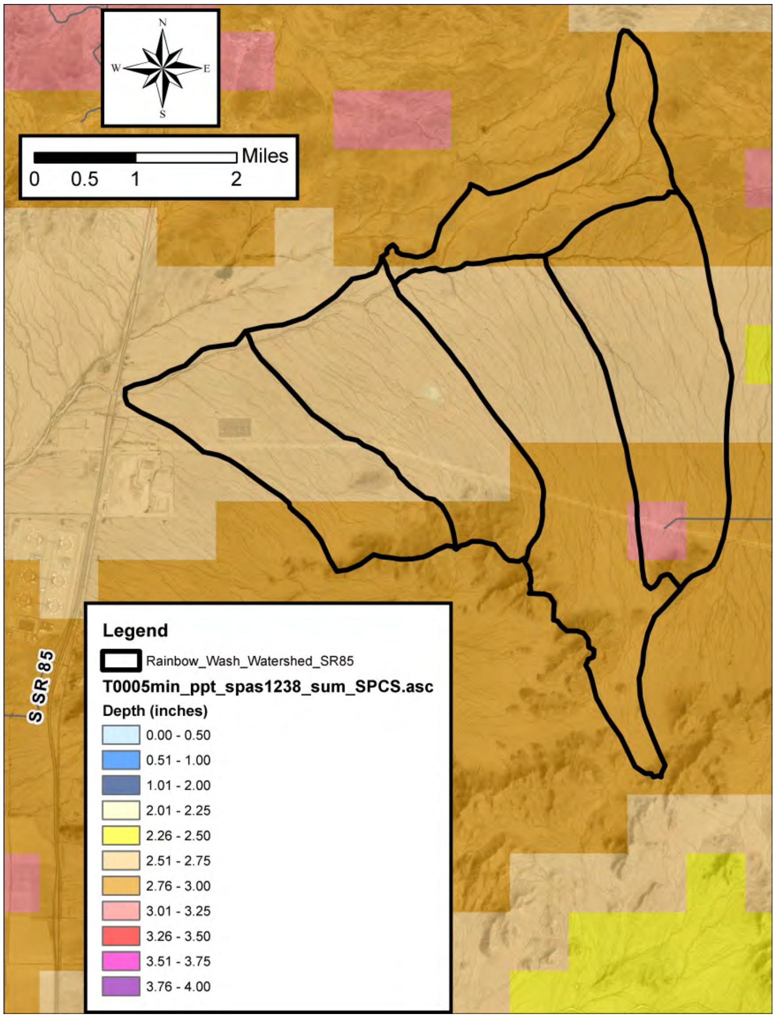
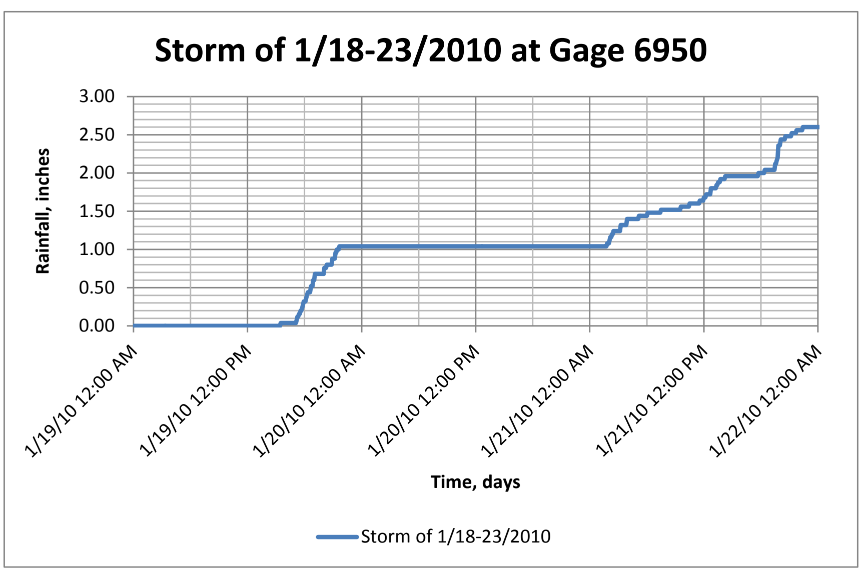


Figure 4.281 Total *GARR* Rainfall for Storm of January 18-23, 2010





Storm of July 29, 2010

The hydrograph results for this storm are shown on Figure 4.282. The total *GARR* rainfall for the storm and the rainfall distribution measured at *FCDMC* Gage 6950 are shown on Figure 4.283. This storm was a long-duration summer storm with a total duration in the area of about 12 hours and a maximum rainfall amount on the watershed of about 2.07 inches, very similar to the August 7-8, 2008 storm. The actual rainfall duration over this watershed was about 4 hours. Note that the rainfall was not uniform over the watershed. Most of the rain occurred along the middle of the north side.

Note that the *FLO-2D* model hydrograph is lower than the measured and the *HEC-1* model barely produces any runoff at all. The *FLO-2D* model initial rising limb and the tail of the receding limb are reasonable matches. There is a possibility that the *GARR* data is in error for this storm. The 1,600 cfs peak discharge seems very high for a rainfall amount of 2 inches only over a portion of the watershed. For this case, if the *GARR* rainfall is considered accurate, *FLO-2D* does provide a reasonable, although low, representation of the storm event. Regardless of the rainfall question, *FLO-2D* provides a better representation of the storm than the *HEC-1* model.

Figure 4.282 Hydrograph Results for Storm of July 29, 2010

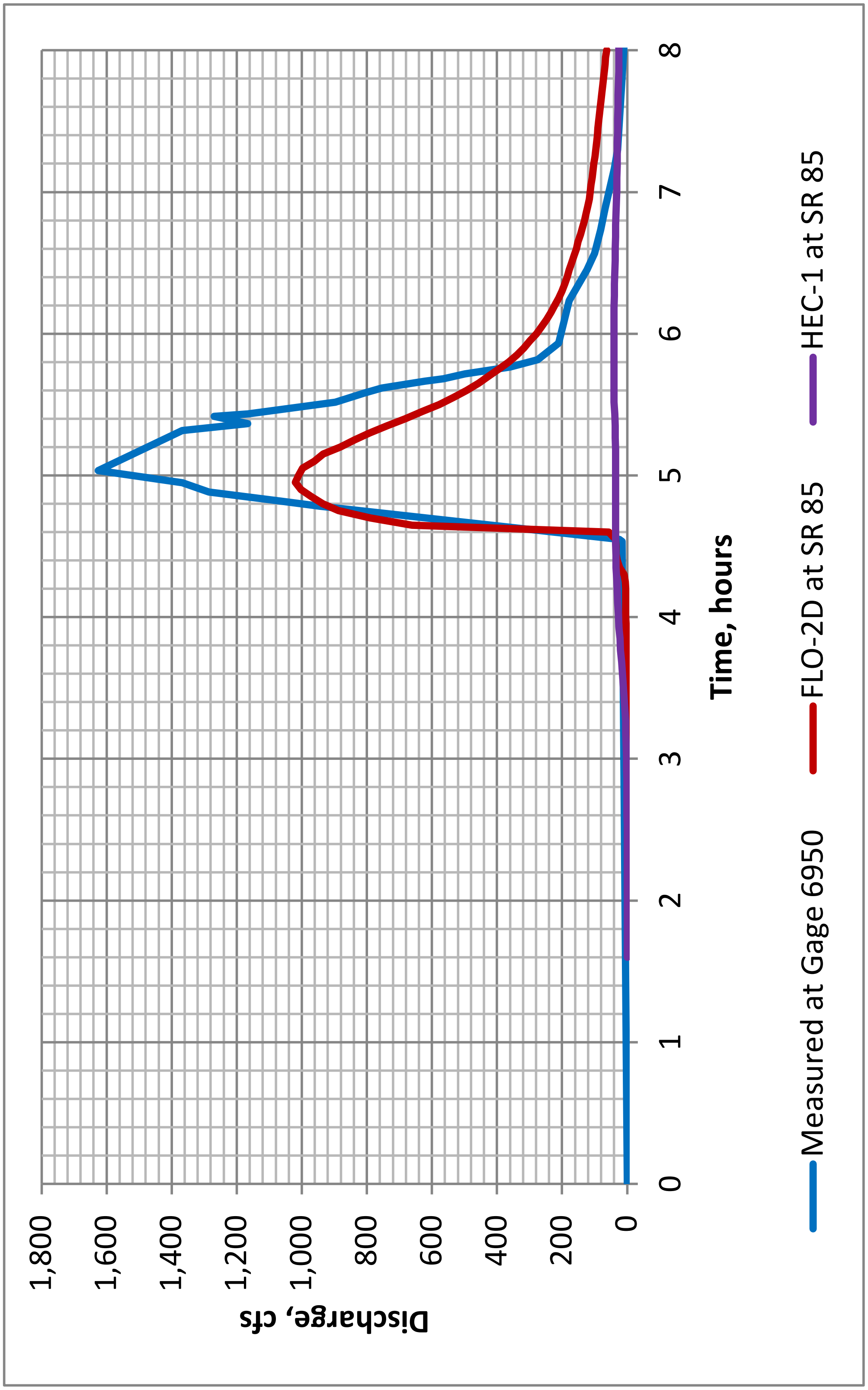
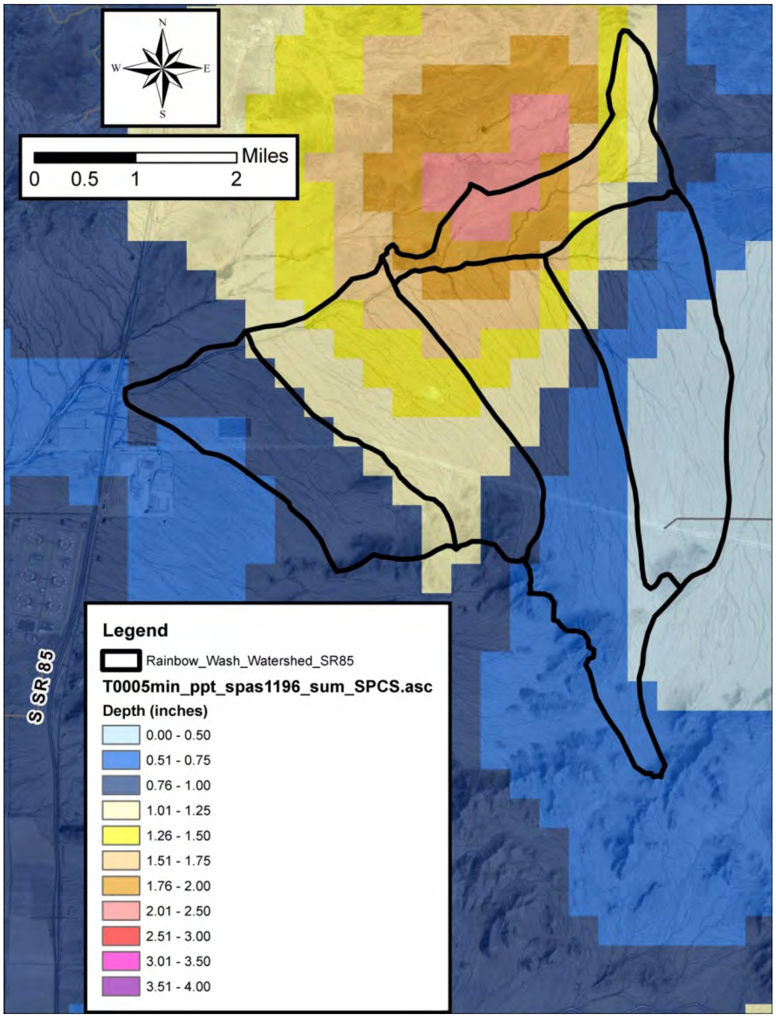
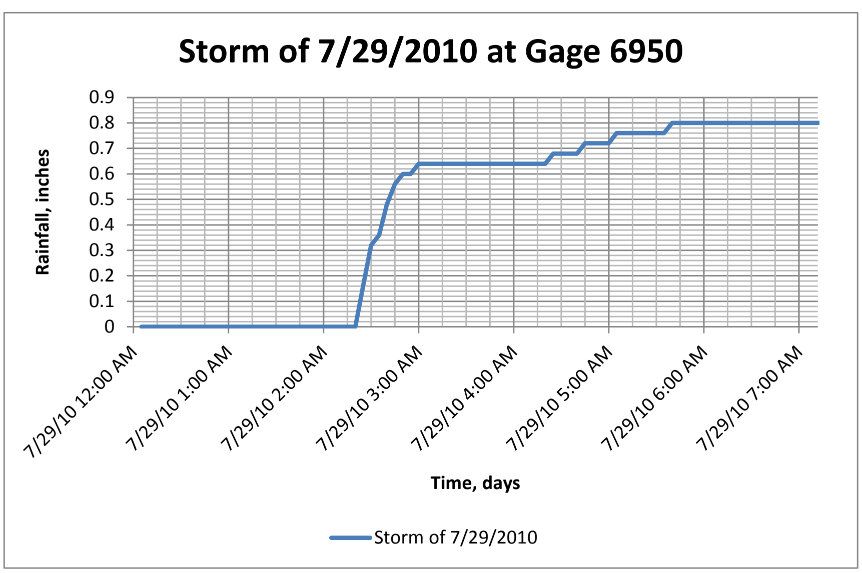


Figure 4.283 Total *GARR* Rainfall for Storm of July 29, 2010





Storm of August 21, 2012

The hydrograph results for this storm are shown on Figure 4.284. The total *GARR* rainfall for the storm and the rainfall distribution measured at *FCDMC* Gage 6950 are shown on Figure 4.285. This storm was a very strong summer storm with a total duration of about 20 hours and a maximum rainfall amount on the watershed of about 3.50 inches. The actual total rainfall duration over this watershed was about 7 hours with the most intense period being about 3 hours. Note that the rainfall was not uniform over the watershed. Most of the rain occurred over the west one half to two thirds of the watershed.

Note that the *FLO-2D* model hydrograph is a close match to the measured. The rising and receding limbs have more volume and similar slopes, but the peak discharge is very close to the measured and the general shape and runoff magnitudes match. The total runoff volume measured at the gage for the first 12 hours was 603 ac-ft, and the runoff volume from the *FLO-2D* model was 654 ac-ft, a reasonable check.

A post-flood aerial photograph (Dec 2012) of Rainbow Wash at SR 85 is shown on Figure 4.286. The estimated flood limits based on a post-flood field reconnaissance and survey of high water marks and post-flood aerial photographs, are also shown. These limits represent the boundary of the flooded areas along Rainbow Wash and the ponding against SR 85. The *FLO-2D* model maximum flood depths are added to the post-flood map as shown on Figure 4.287. The *FLO-2D* maximum depth results compare very well with the estimated actual flood limits.

The *HEC-1* model provides a less reasonable representation and the peak discharge and the runoff volume is quite a bit lower than measured. For this case, *FLO-2D* provides a good representation of the storm event within the possible range of error, and a better representation than the *HEC-1* model.

Figure 4.284 Hydrograph Results for Storm of August 21, 2012

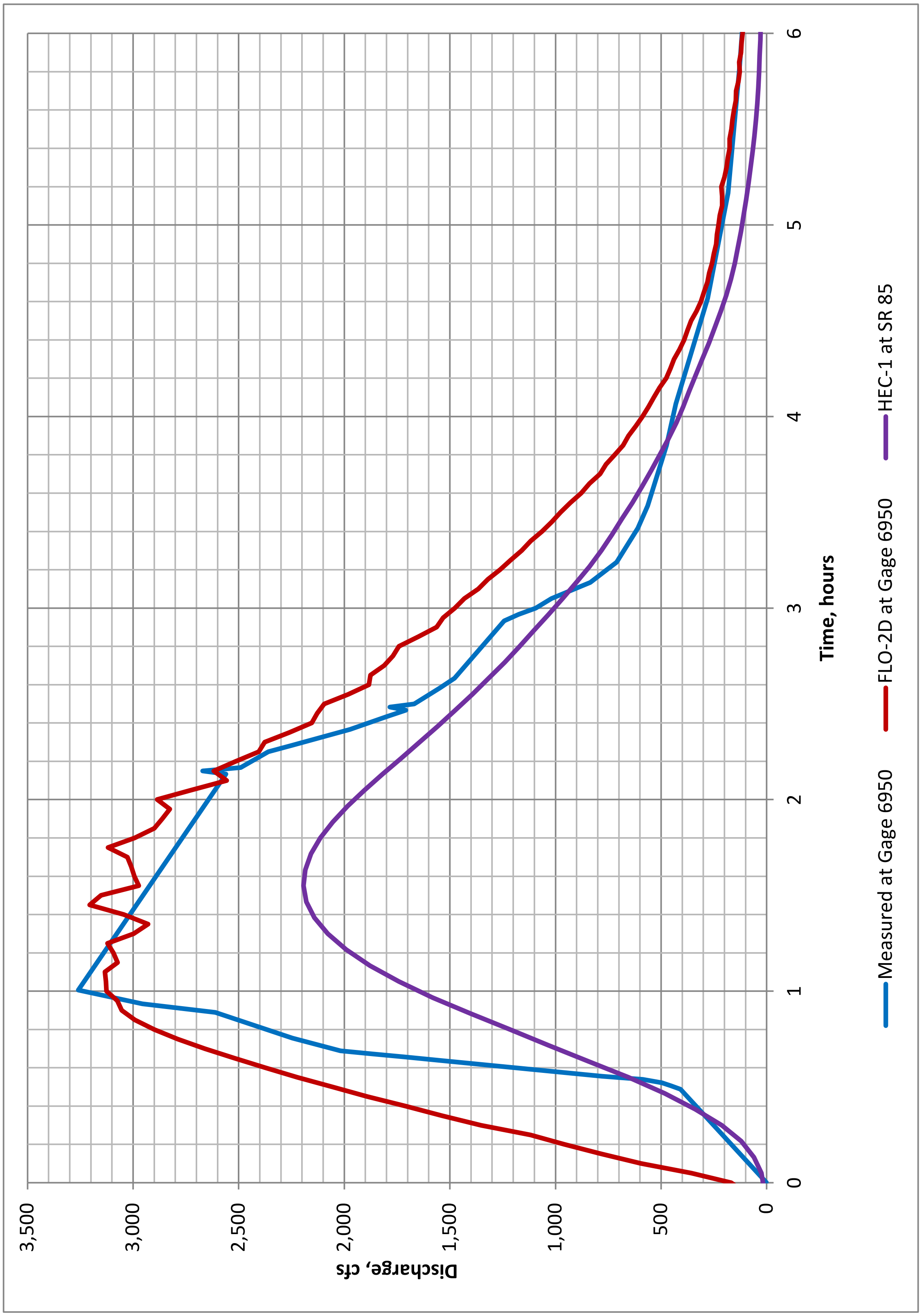
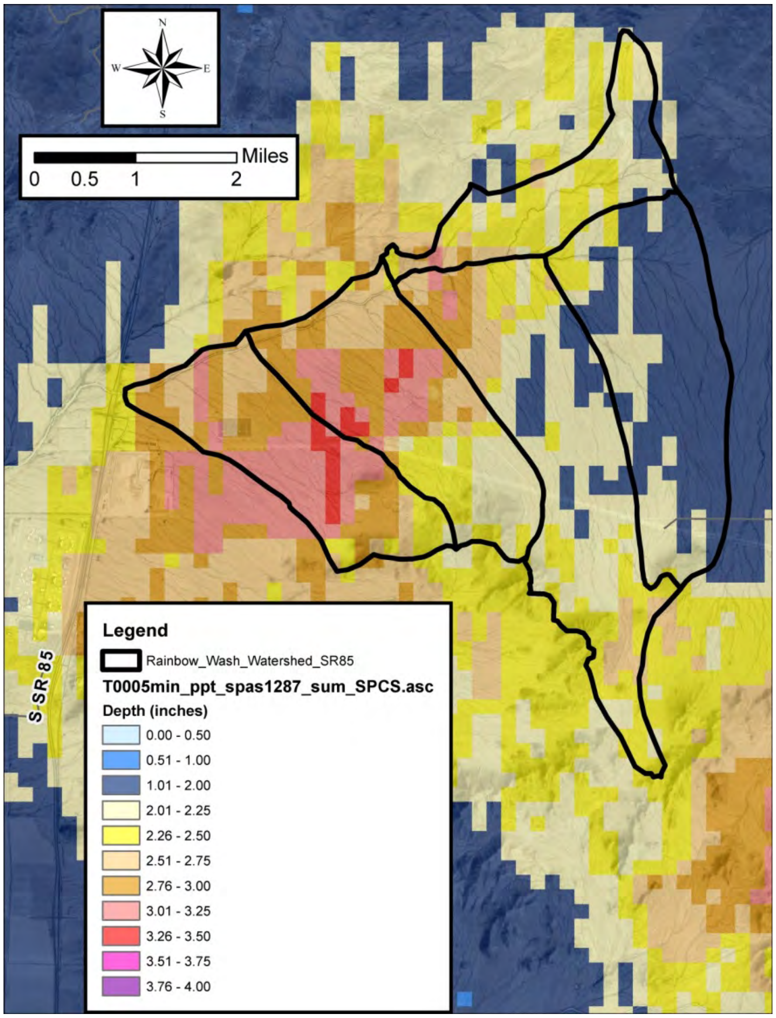


Figure 4.285 Total *GARR* Rainfall for Storm of August 21, 2012



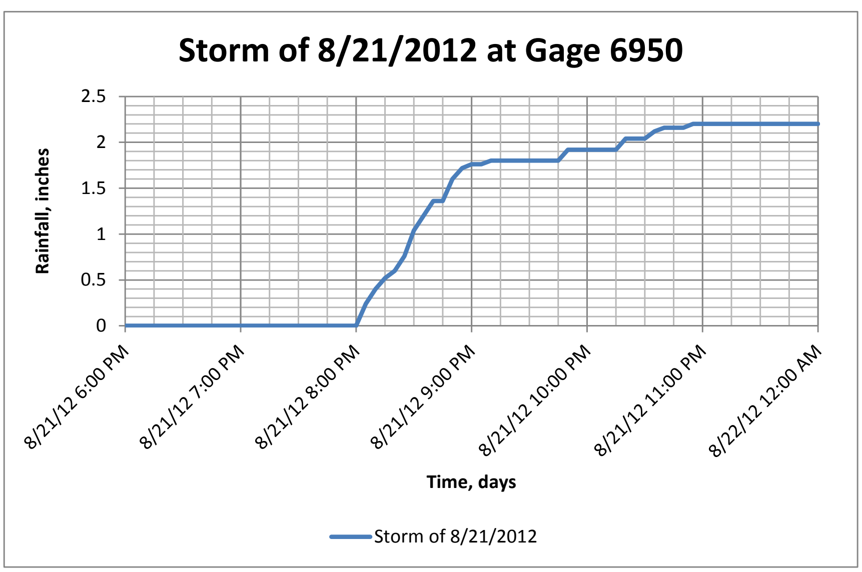


Figure 4.286 December 2012 Aerial Photograph – Post-Flood

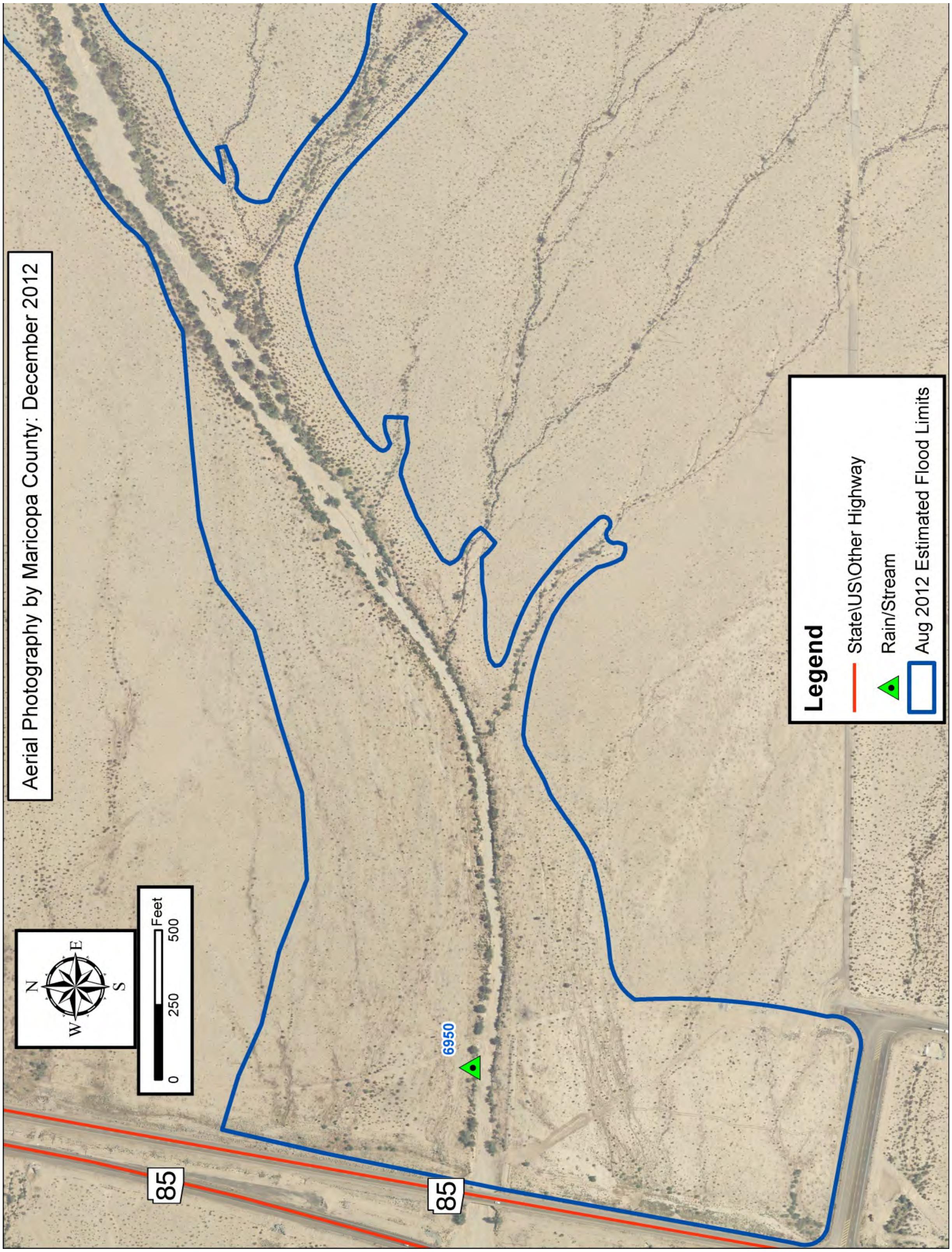
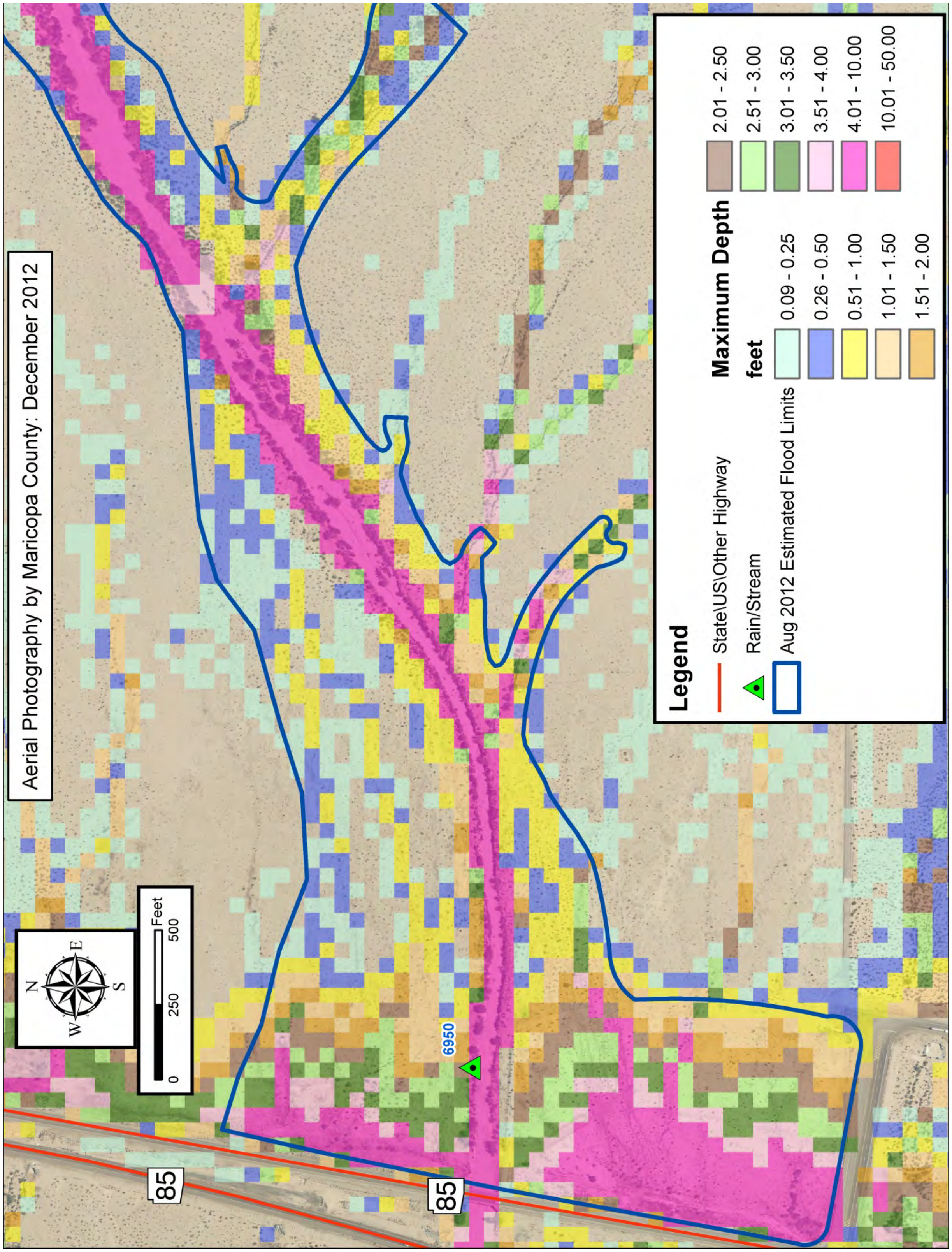


Figure 4.287 *FLO-2D* Maximum Depth Results



Discussion

Modeling known storm events with a hydrologic model is always problematic. There are many variables with a fairly high degree of uncertainty for each, including: 1. Spatial and temporal variability of rainfall, 2. Soils information based on a very limited number of physical samples, 3. Infiltration estimation method that is a numerical simplification of the actual physical process, and 4. Uncertainties in hydraulic modeling parameters used to prepare the stream flow gage rating curve. With these uncertainties in mind, the Rainbow Wash *FLO-2D* storm replication test models do an admirable job of reproducing the storm events when compared with the gage-measured flow rates. All four test cases provide better results than the District's current *1D* modeling method. Based on these tests, the *FLO-2D* model is capable of reasonably representing actual flood events.

4.3.1.3 Seven Springs Wash Test Cases

General

The Seven Springs Wash in Maricopa County was selected for a storm reconstitution test case. The location of the Seven Springs Wash watershed is shown on Figure 4.14. The vicinity map for the watershed is shown on Figure 4.288. The watershed for Seven Springs Wash at North Seven Springs Road and National Forest Road 254 is shown on Figure 4.289. The watershed area is approximately 8.02 square miles. *FCDMC* owns and maintains a recording pressure tranducer stream flow gage (*FCDMC* Gage 4963), which has been in operation since March 12, 2002. The *FCDMC* also owns and operates three rain gages in and adjacent to the watershed: Gage 4940 Humboldt Mountain, Gage 4690 Seven Springs Wash, and Gage 5955 Camp Creek, as shown on Figure 4.289.

The wash is a well-defined natural channel with a dominately trapezoidal cross section, a bed of sand, gravel, and large cobbles underlain by rock outcrop and caliche in various locations, and well vegetated banks. Refer to photographs of the wash at the gage site shown on Figure 4.290.

Several major flows have been recorded at this gage site in recent years. Two (2) storms were selected as test cases and are listed in Table 4.45. The 100-year 24-hour storm design rainfall was also modeled.

Figure 4.288 Seven Springs Wash Vicinity Map

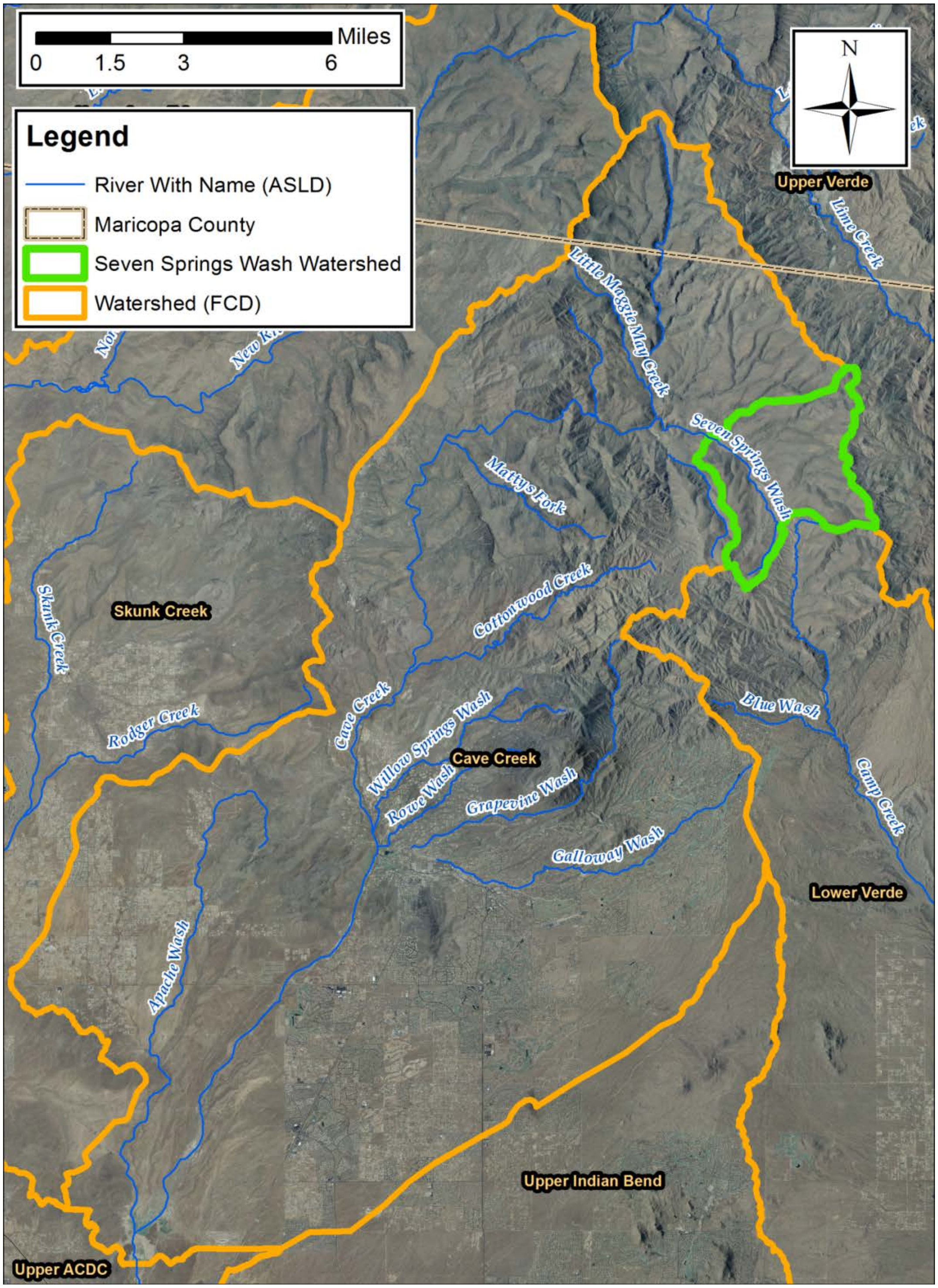


Figure 4.289 Seven Springs Wash Watershed Map

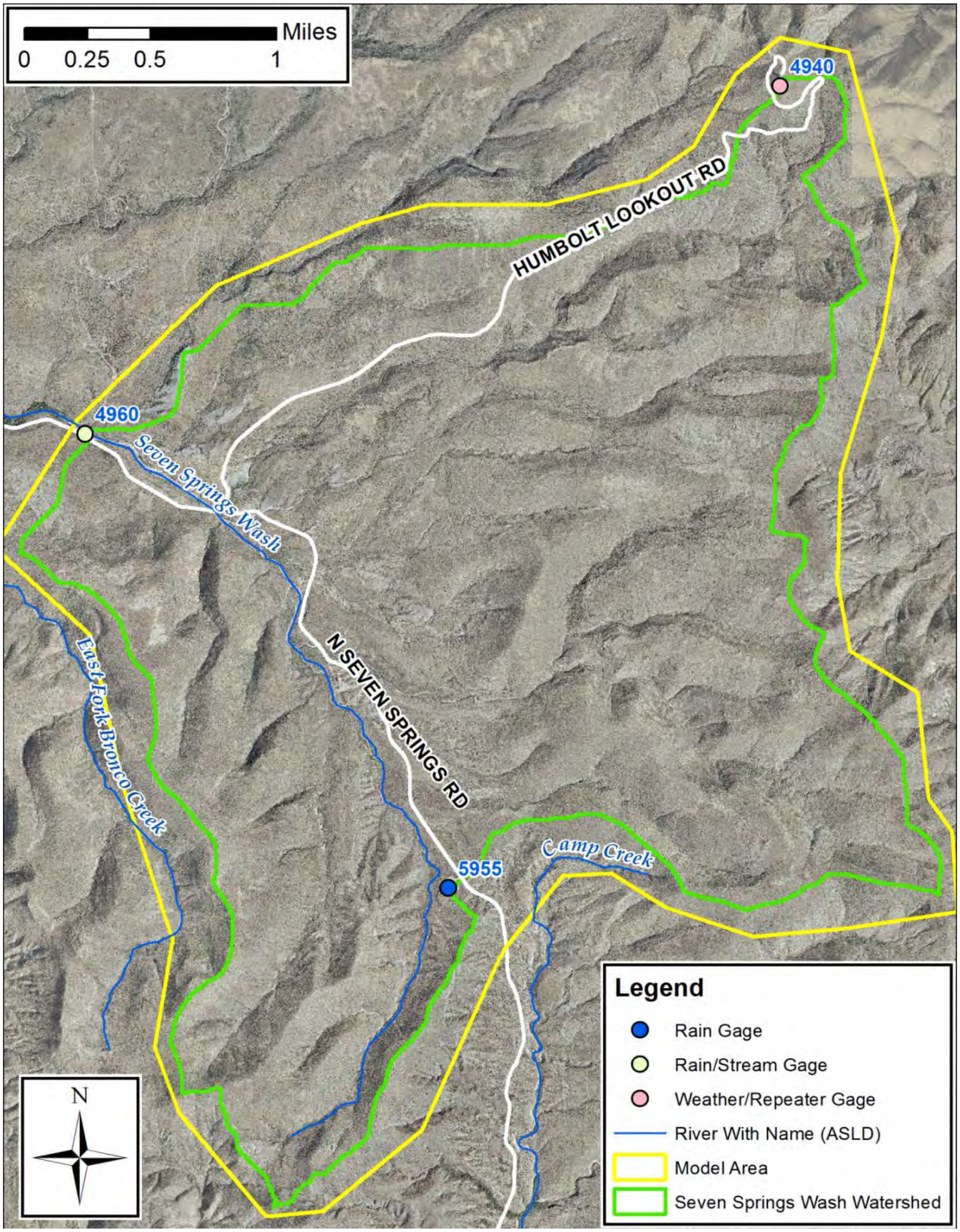


Figure 4.290 FCDMC Seven Springs Wash Gage Site Photographs

Looking at sensor location

Looking downstream from gage sensor

Table 4.45 Seven Springs Wash Test Case Storms

Storm Date	Maximum Gage Height, ft Gage 4963	Estimated Peak Discharge, cfs
(1)	(2)	(3)
July 29 – 30, 2006	7.38	2,464
July 14, 2008	2.80	200
100-year 24-hour Design Storm (*FLO-2D*)	11.51	7,900

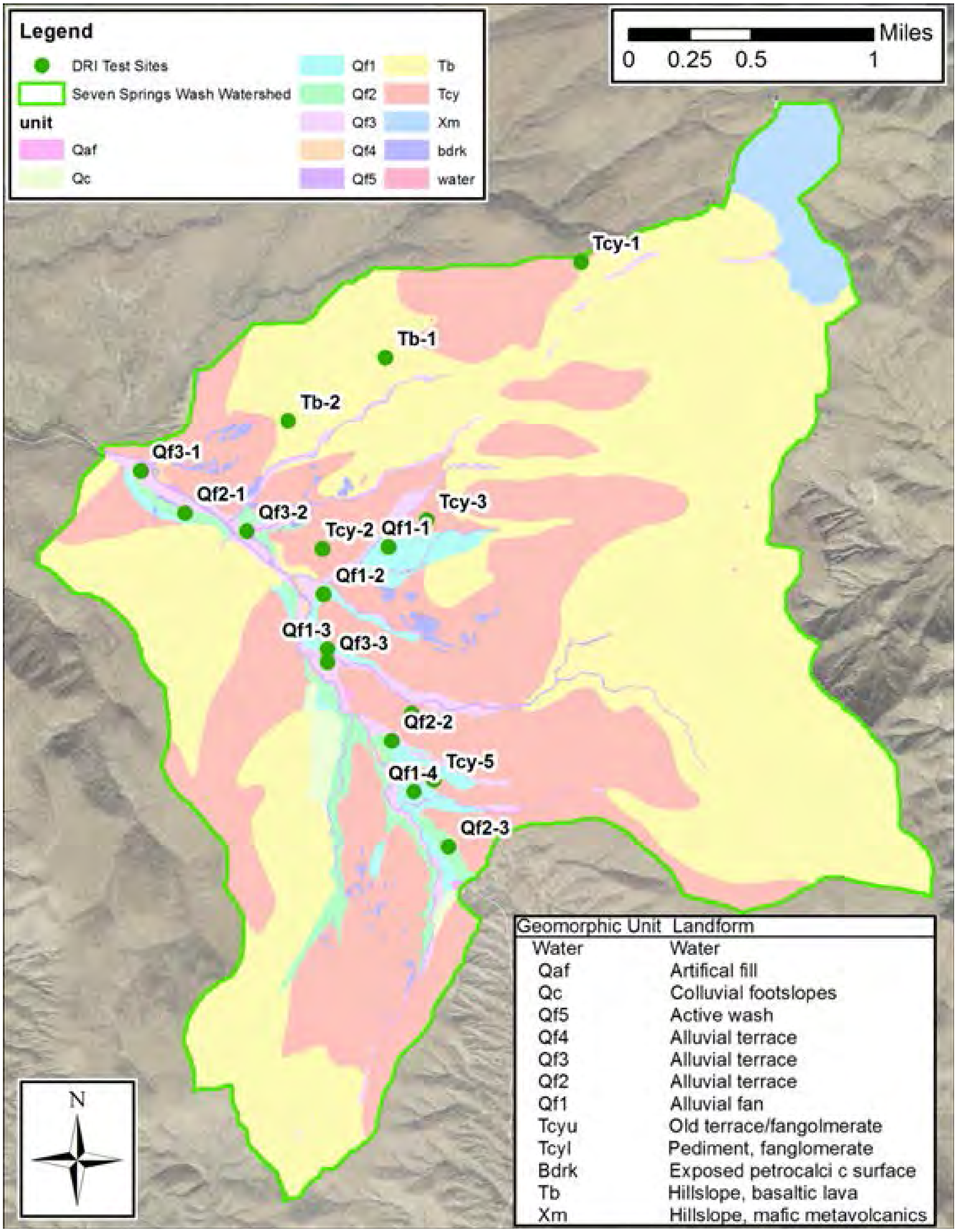
The *FLO-2D* model used to simulate these storms was developed in-house for the purposes of this study. The model development and parameters are documented in *Seven Springs Wash Two-Dimensional Modeling Technical Data Notebook* (FCDMC, 2015a). The key information for the model of the Seven Springs Wash Watershed includes:

- *FLO-2D* Pro Build 14.08.09
- 20-foot grid size.
- Topography based on the FCDMC 10-foot contour interval digital terrain model, flight date 2001.
- *G&A* rainfall loss method using field measured parameters from *DR*I (Chen, Miller, Bacon, & Forsee, 2012).

Test locations for field measured *G&A* parameters were sited within the geomorphic units shown on Figure 4.291. The geomorphic assessment, field testing, and generation of parameters is documented in

<u>*Terrain, Soils and Runoff Potential in the Seven Springs Wash Watershed in Maricopa County*</u> (Chen, Miller, Bacon, & Forsee, 2012).

Figure 4.291 Seven Springs Wash Geomorphic Units



Storm *GARR* data was generated by METSTAT, Inc. for each storm as briefly described in Section 4.3. Refer to the following reports for documentation of the generation of the GARR data:

6. *Storm Precipitation Analysis Report, Storm of July 29-30, 2006, Maricopa County, Arizona, SPAS Storm #1239* (METSTAT, Inc., 2012d).
7. *Storm Precipitation Analysis Report, Storm of July 13-14, 2008, Maricopa County, Arizona SPAS Storm #1240* (METSTAT, Inc., 2012e).

The *GARR* data was used to create the *FLO-2D* RAINCELL.DAT input data file, which simulates the actual moving storm rainfall using 5-minute time intervals. The resulting runoff hydrograph for each storm was then compared with the measured hydrograph from *FCDMC* Gage 4963.

The *HEC-1* model from the *FEMA* Flood Insurance Study (*FIS*) for Seven Springs Wash was also used to model each storm. The 5-minute rainfall data was averaged within each model sub-basin to create an individual rainfall distribution for every sub-basin. The model was also revised using the *DRI* field measured *G&A* infiltration parameters, which were also used in the *FLO-2D* model. Refer to *Cave Creek Above Carefree Highway Floodplain Delineation Study, FCD 95-28, Technical Data Notebook, Hydrology, Existing Condition* (GVSCE, 1997) for the supporting information for the *FIS HEC-1* model. The modifications to the *HEC-1* model for this study are documented in (FCDMC, 2015a). The results are presented and discussed in the following sections. Note that the time base of the *FLO-2D* and *HEC-1* models were shifted to better align the model results with the measured for comparison purposes.

Storm of July 29-30, 2006

The hydrograph results for this storm are shown on Figure 4.292. The total *GARR* rainfall for the storm and the rainfall distribution measured at *FCDMC* Gages 4940, 4960 and 5955 are shown on Figure 4.293. This was a relatively short duration summer storm with the majority of the rain falling in the first three (3) hours. The maximum rainfall amount on the watershed was about 3.04 inches. Note that the rainfall was not uniform over most of the watershed, with the majority of the rain along the middle of the watershed in a northwest to southeast band, and lower rainfall totals outside of that band.

Note from reviewing Figure 4.292 that the *FLO-2D* model results are a fairly close match to the measured. The *FLO-2D* hydrograph rising limb is a very good match with measured. The *FLO-2D* peak discharge is lower than measured but a good match when the measured oscillations are averaged out as shown by the moving average hydrograph. The receding limb of the *FLO-2D* hydrograph has less volume than the measured. *FLO-2D* is producing slightly less runoff volume than measured for this case: 162 ac-ft for *FLO-2D* compared with 168 ac-ft measured. The *HEC-1* model does not match the measured. The peak discharge is about 30% lower than measured, the runoff volume about 23% higher, and the hydrograph shape does not match. The averaging of the rainfall within each sub-basin results in the loss of high intensity rainfall over smaller areas. The *FLO-2D* model does a better job of representing the high intensity rainfall areas. For this case, *FLO-2D* provides a reasonable representation of the storm event, and a much better representation than the *HEC-1* model. The possible errors for the measured hydrograph include measurement error associated with the stream flow gage, and the numerous possible errors associated with the hydraulic model the gage rating curve is based on. As with the Rainbow Wash storm events, the possible *FLO-2D* and *HEC-1* model errors include a plus or minus 10% to 20% error range for the *GARR* data, as well as the normal errors for models of these types.

Storm of July 14, 2008

The hydrograph results for this storm are shown on Figure 4.294. The total *GARR* rainfall for the storm and the rainfall distribution measured at *FCDMC* Gages 4940, 4960 and 5955 are shown on Figure 4.295. This was a short duration summer storm with the majority of the rain falling in the first one (1) hours. The maximum rainfall amount on the watershed was about 2.43 inches. Note that the rainfall was not uniform over the watershed, with the heaviest rain occurring at the bottom of the watershed. Very little rain occurred in the upper watershed.

Note from reviewing Figure 4.294 that the *FLO-2D* model results are of similar magnitude to the measured. The *FLO-2D* hydrograph rising limb is a very good match with measured. The *FLO-2D* peak discharge matches measured. The receding limb of the *FLO-2D* hydrograph has significantly more volume than the measured. *FLO-2D* is producing significantly more runoff volume than measured for this case: 29.1 ac-ft for *FLO-2D* compared with 11.5 ac-ft measured. The *HEC-1* model does not match the measured. The peak discharge is about 280% higher than measured, the runoff volume (115 ac-ft) about 10 times higher, and the hydrograph shape does not match. HEC-1 is significantly underestimating infiltration for this storm. For this case, *FLO-2D* provides a reasonable representation of the storm event, and a much better representation than the *HEC-1* model. The possible errors for the measured hydrograph include measurement error associated with the stream flow gage, and the numerous possible errors associated with the hydraulic model the gage rating curve is based on. As with the Rainbow Wash storm events, the possible *FLO-2D* and *HEC-1* model errors include a plus or minus 10% to 20% error range for the *GARR* data, as well as the normal errors for models of these types.

Figure 4.292 Hydrograph Results for Storm of July 29-30, 2006

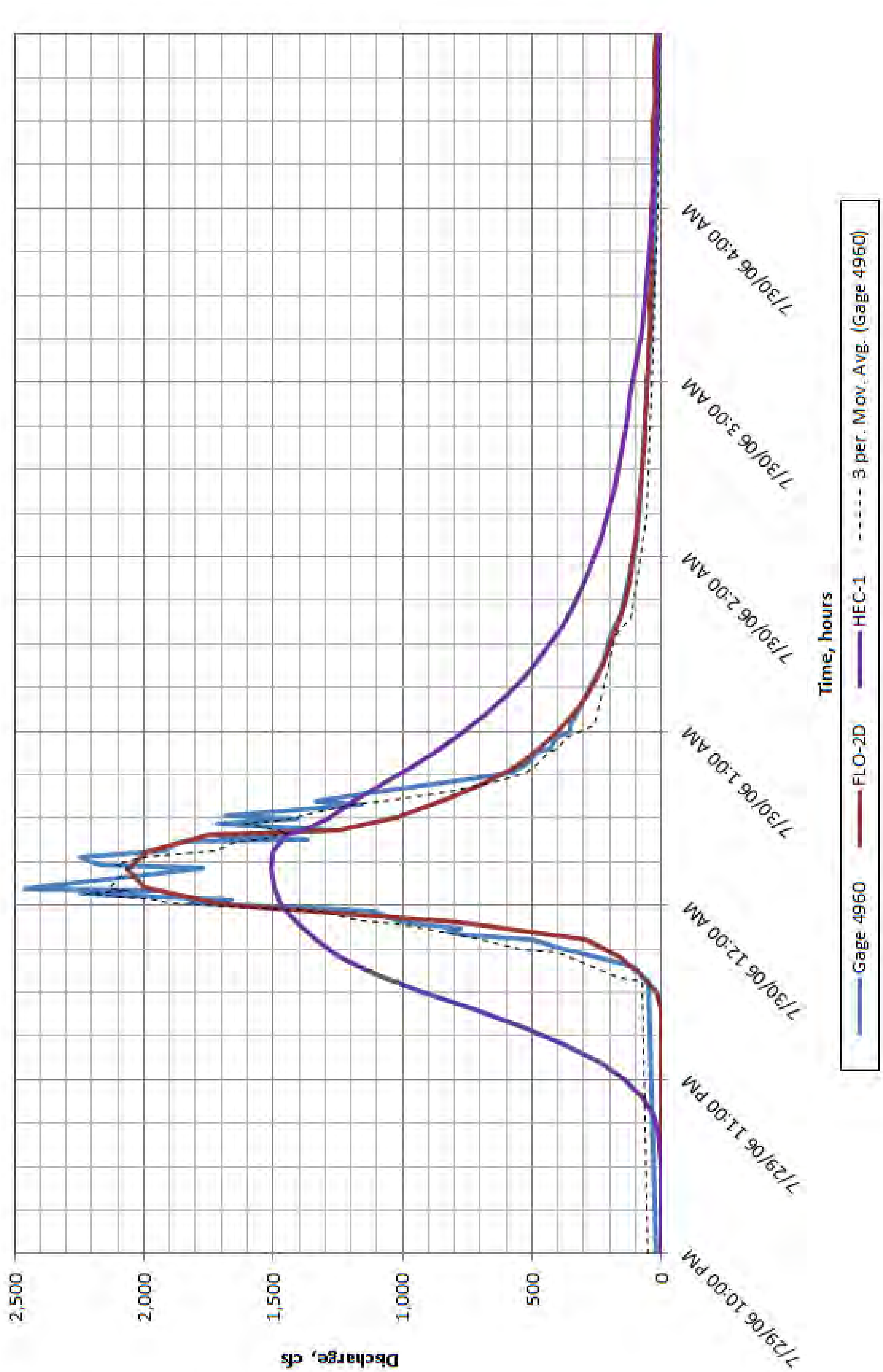
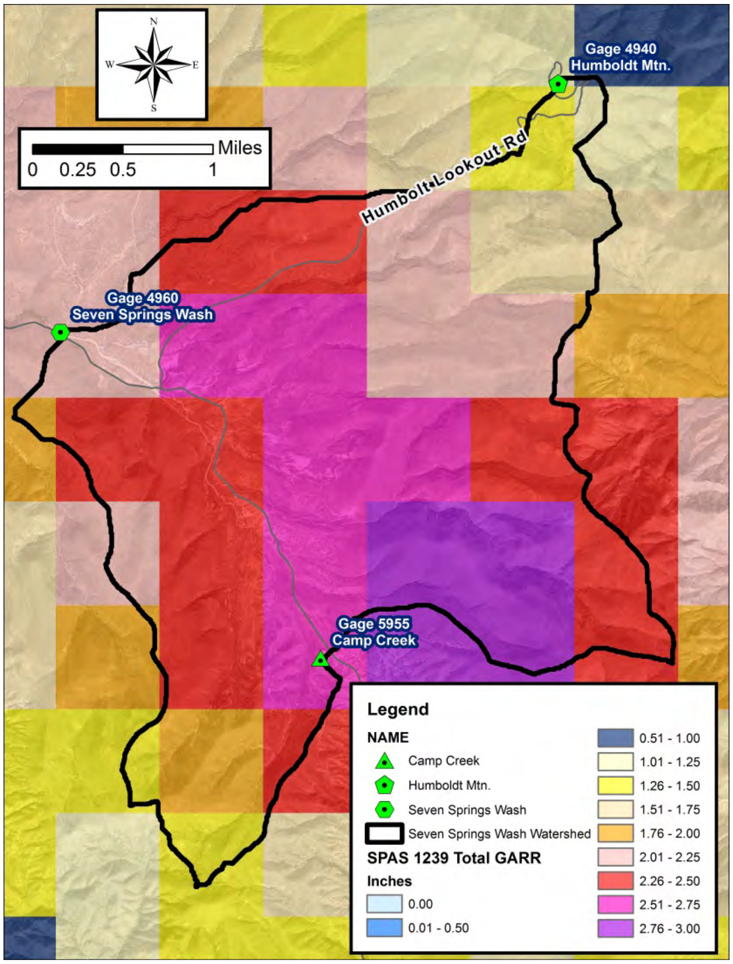


Figure 4.293 Total *GARR* Rainfall for Storm of July 29-30, 2006



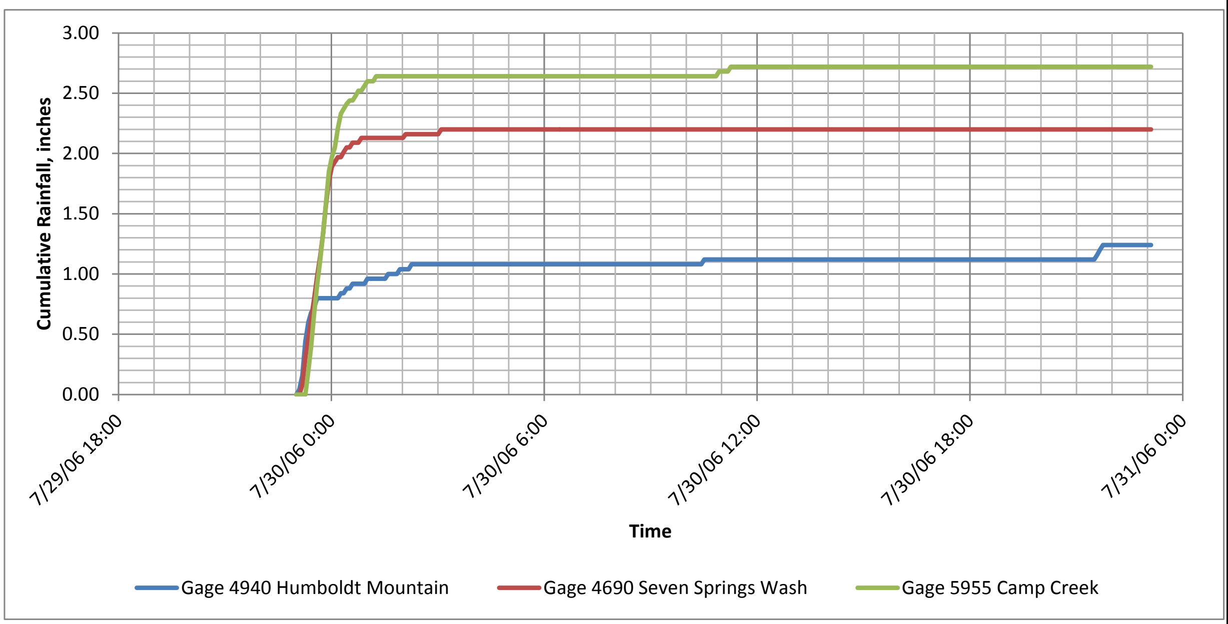


Figure 4.294 Hydrograph Results for Storm of July 14, 2008

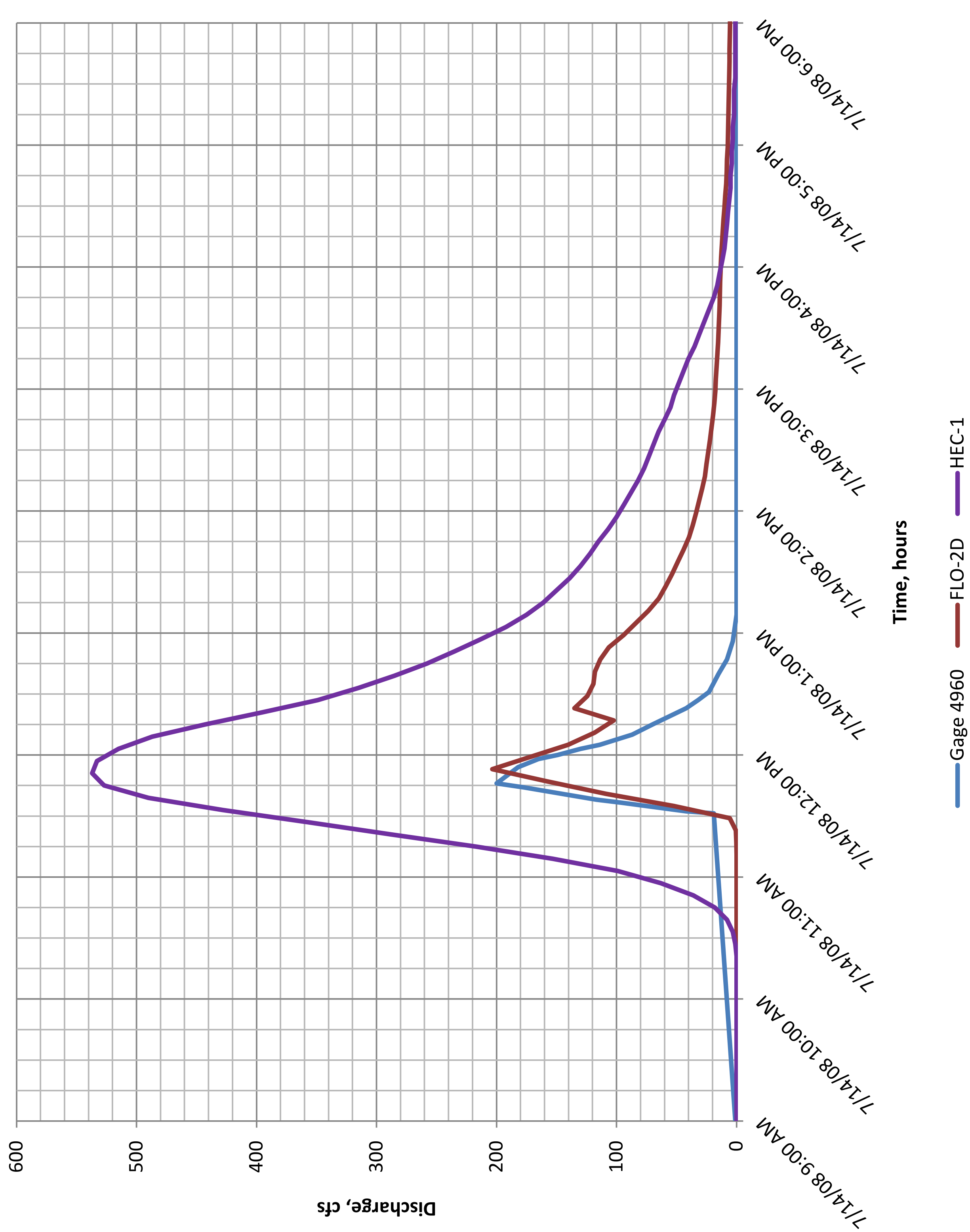
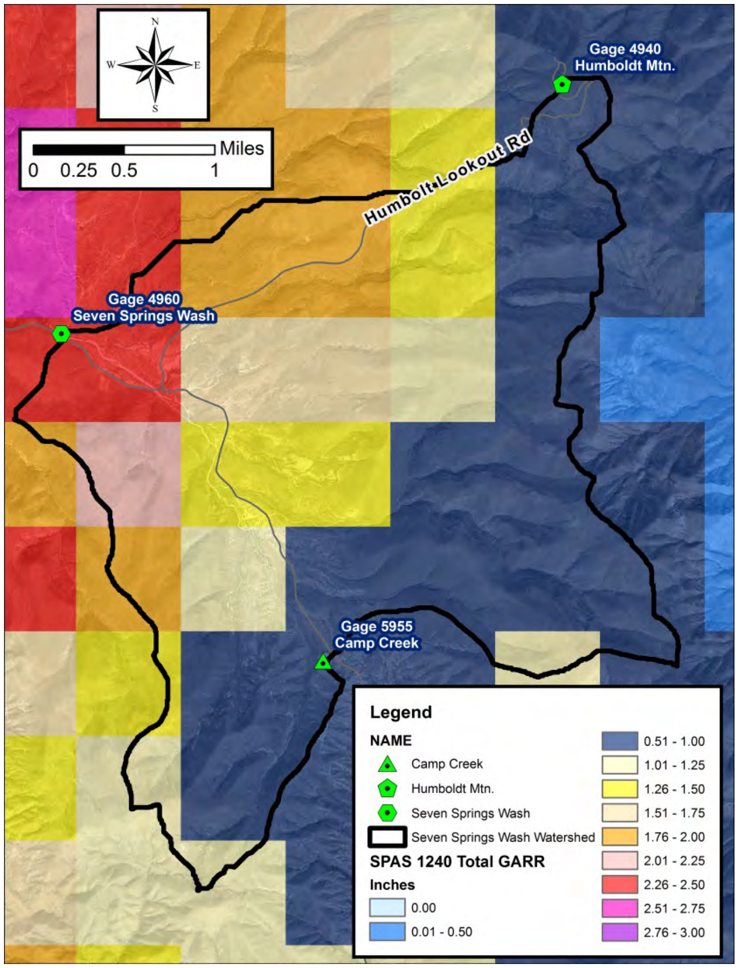
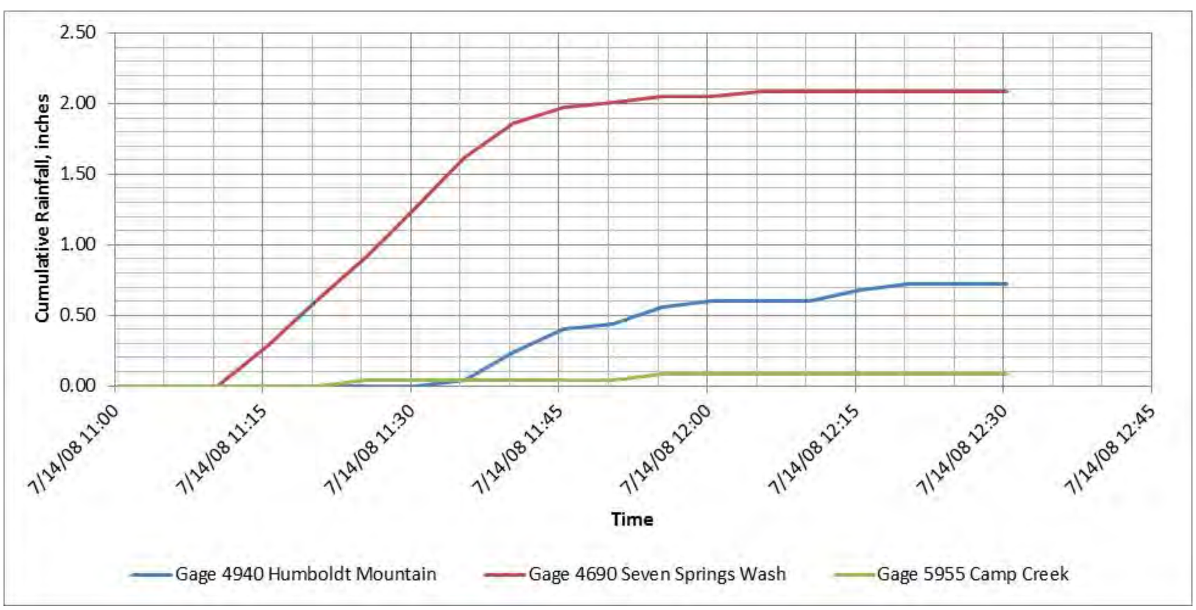


Figure 4.295 Total *GARR* Rainfall for Storm of July 14, 2008





<u>100-year 24-hour Design Storm</u>

This test case compares *FLO-2D* with *HEC-1* for a standard *FCDMC* 100-year 24-hour design storm of the Seven Springs Wash watershed. The *FLO-2D* model is the same model used for the 2006 and 2008 actual storms described previously. The difference is that the *NRCS* Type II 24-hour rainfall distribution was used with a point rainfall depth of 4.67 inches instead of the RAINCELL.DAT option. The 4.67 inch rainfall depth is the depth-area reduced value for an 8 square mile watershed. Refer to Chapter 2 of the *DDM Hydrology* (FCDMC, 2013a) for more information on depth-area reduction and the *NRCS* Type II rainfall distribution.

The *HEC-1* model is similar to that used for the 2006 and 2008 actual storms described previously. The difference is the model was converted to apply the 'JD' record option, also using the *NRCS* Type II rainfall distribution, and a point rainfall depth value of 4.98 inches for 0.01 square miles.

The purpose of this comparison is to contrast the model results between a *1D* and *2D* model when the *FCDMC* standard design storm is applied. The resulting runoff hydrographs at Gage 4960 are shown on Figure 4.296. The total runoff volume from the *FLO-2D* model is 644 ac-ft as compared to 860 ac-ft from the *HEC-1* model. The *FLO-2D* runoff volume is expected to be less than *HEC-1* due to transmission losses that are not included in the *HEC-1* model. The limiting infiltration depth option was applied in the *FLO-2D* model using the same values developed for modeling the 2006 and 2008 historic storms. This is assumption made for the purposes of this report but the user may need to increase the LID estimates if the design storm rainfall volumes are significantly different than the real storm volumes. Engineering judgment will be required.

Even though the *HEC-1* model produces more runoff volume than *FLO-2D* for this case, the *FLO-2D* peak discharge is significantly higher: 7,600 cfs for *FLO-2D* vs 4,800 cfs for *HEC-1*. After examination of the model results for the July 2006 storm, it is expected that *FLO-2D* would produce a much higher peak discharge than *HEC-1* for a major storm. The leading edge of the *FLO-2D* hydrograph rising limb has virtually no runoff when compared with *HEC-1*, which has a gradually increasing discharge. The *FLO-2D* result is reasonable when transmission losses are considered. The rising limb resulting from the most intense period of rainfall is nearly identical for the two models except that *FLO-2D* has a much higher peak discharge. On the receding limb, *FLO-2D* has significantly less volume and is much steeper than *HEC-1*, again due to accounting for transmission losses. However, *FLO-2D* has a higher discharge than HEC-1 for the receding tail after hour 16.5 because the limiting infiltration depths have been met in most of the lower watershed wash bottoms. Refer to Figure 4.297.

An additional check for reasonableness is to apply indirect methods 2 and 3 from the *DDM Hydrology*. Refer to Figure 4.298 and Figure 4.299. The Method 2 comparison, which includes 100-year peak discharge estimates for USGS gages in Arizona, shows the *HEC-1* result within the 75% tolerance limits and the *FLO-2D* result being just outside the 75% tolerance limits. Both are within the largest cluster of points for watersheds near the size of the Seven Springs Wash watershed.

Figure 4.296 Hydrograph Results for 100-year 24-hour Design Storm

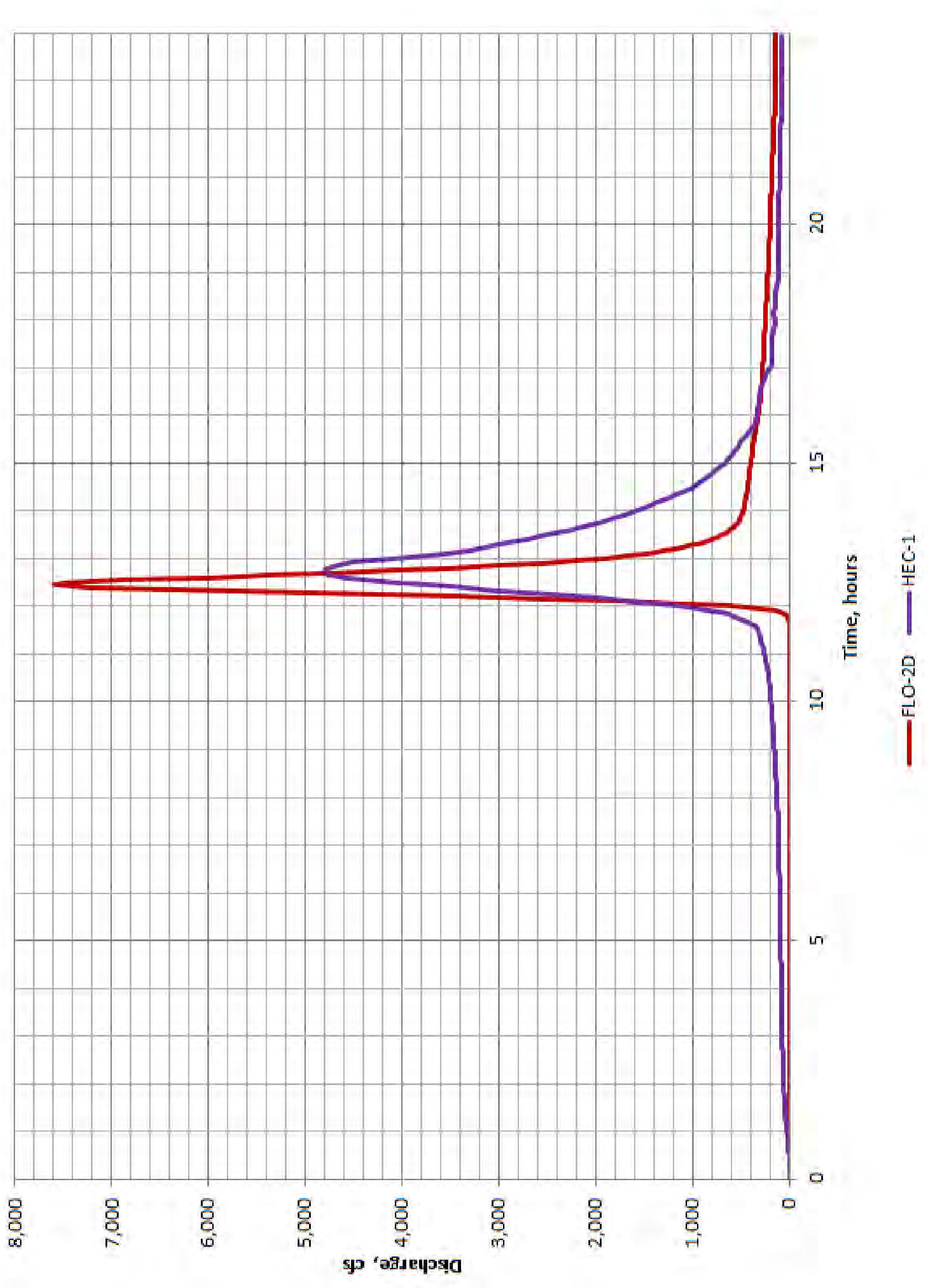


Figure 4.297 Grids Where Limiting Infiltration Depth is Reached

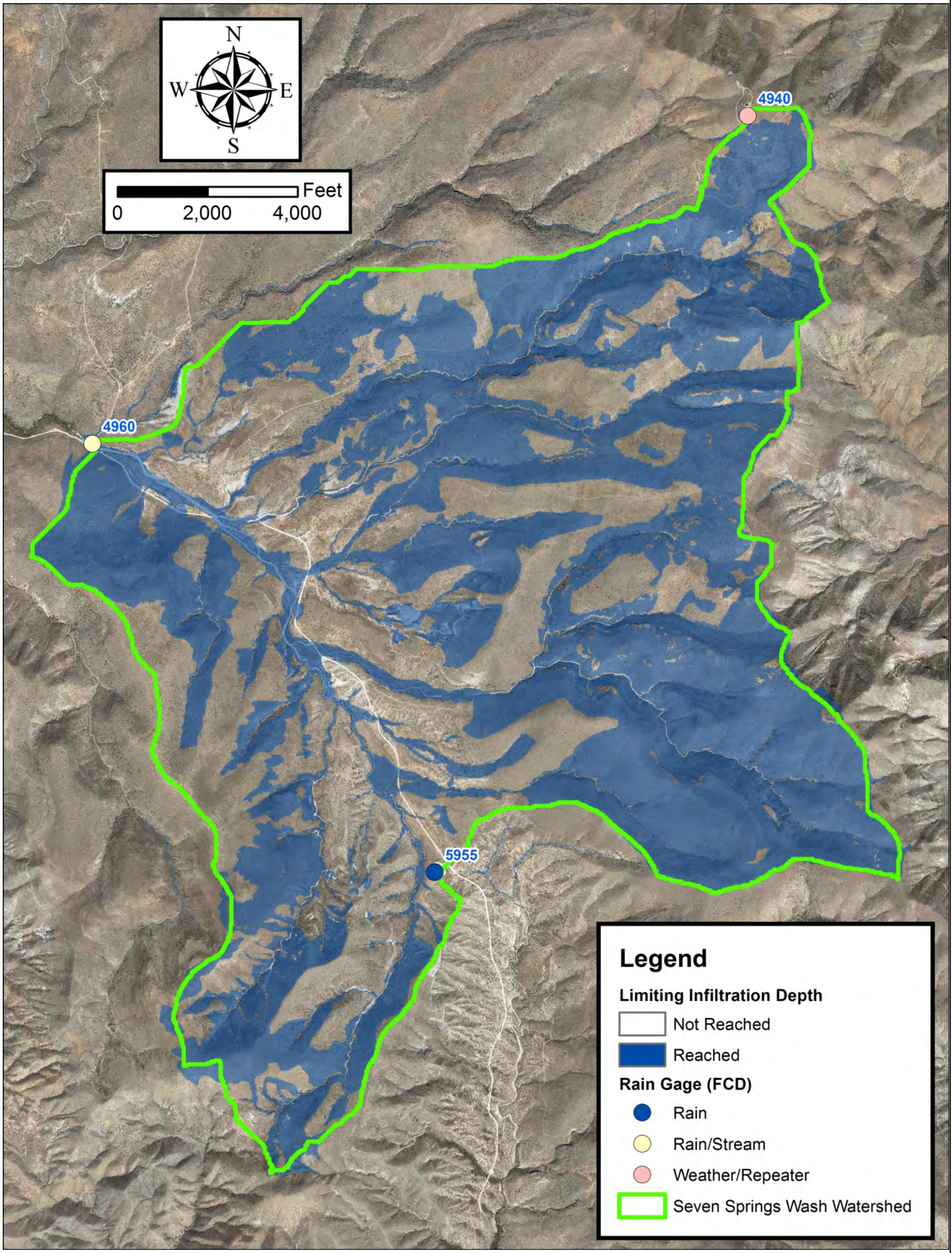


Figure 4.298 Indirect Method 2 Comparison

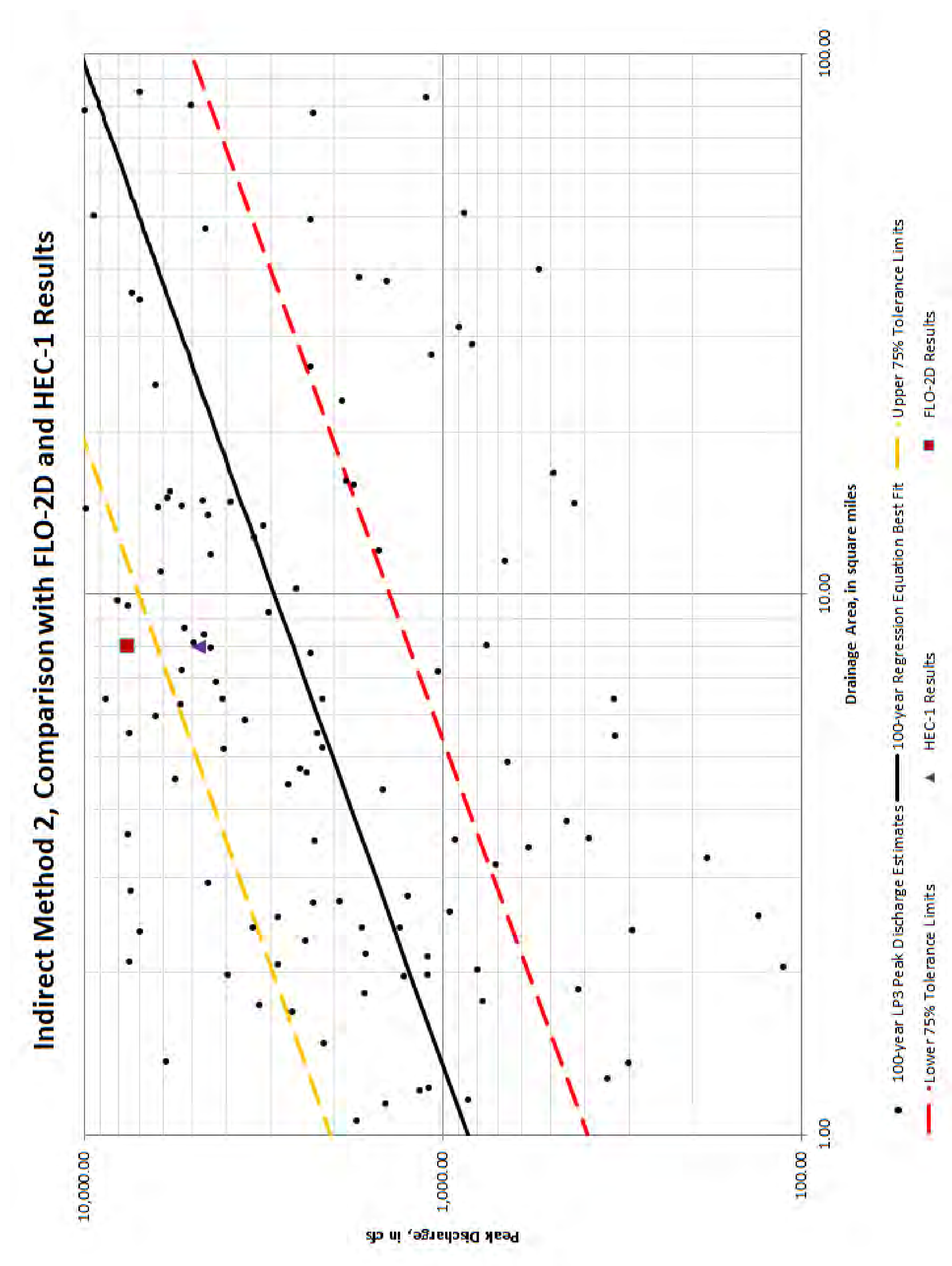
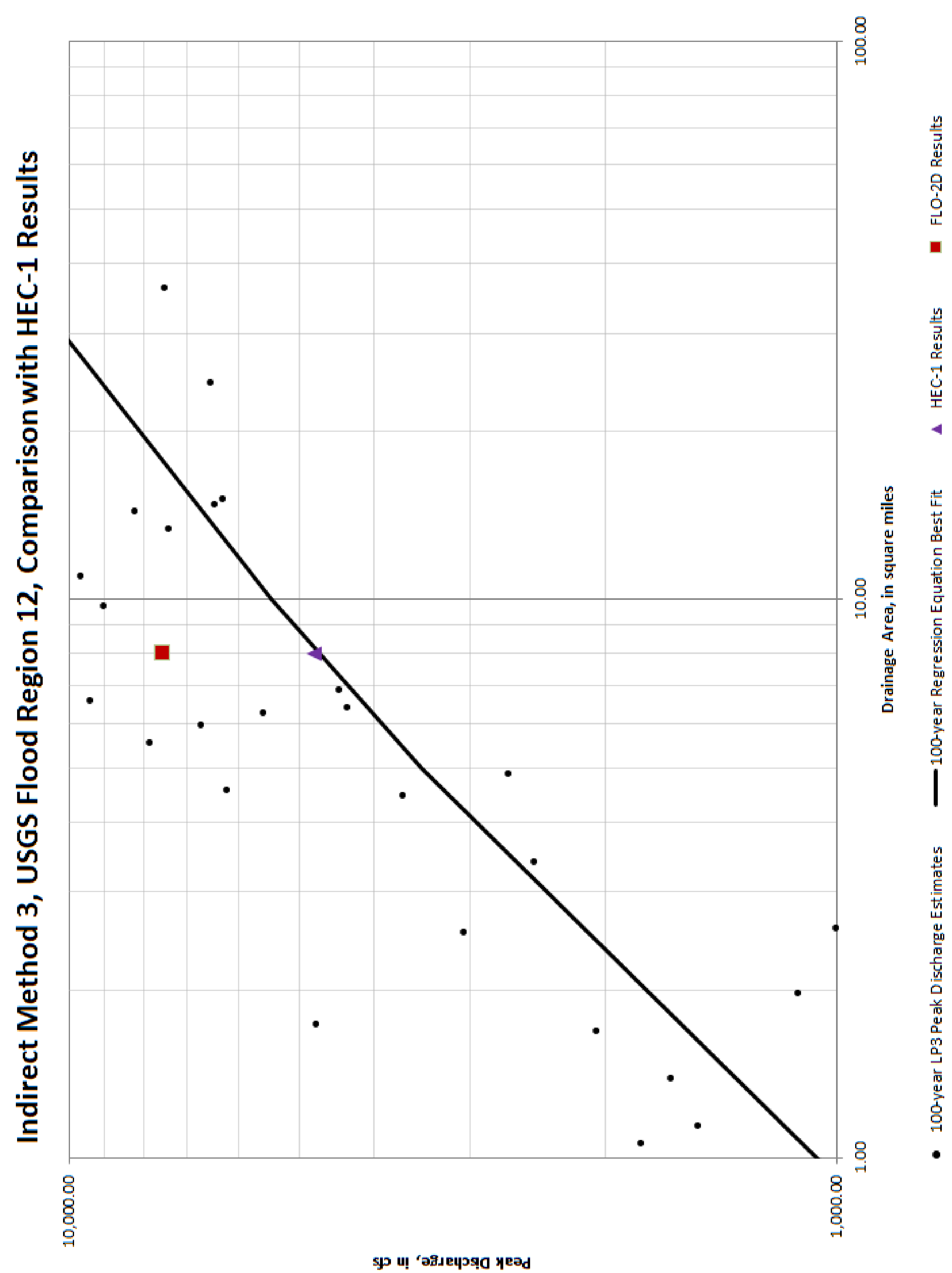


Figure 4.299 Indirect Method 3 Comparison



The Method 3 comparison, which is the *USGS* Regional Regression Equation for Region 12 and supporting point data, shows the *HEC-1* result right on the regression equation line and the *FLO-2D* result centered within the cluster of data points for watersheds near the size of the Seven Springs Wash watershed. Based on these comparisons, the *FLO-2D* peak discharge result is not unreasonably high, and fits better with the 100-year gage estimates for similar sized watersheds than the *HEC-1* result.

For a final check for reasonableness, the historic record for Gage 4963 was examined. The measured peak discharges by water year from the *FCDMC* web site are listed in Table 4.46 (http://alert.fcd.maricopa.gov/alert/Flow/4963.htm). Note that the peak discharge of record, 11,700 cfs, occurred in 2005. This was the same year as a forest fire that occurred on the 7 Springs Wash watershed, so the peak discharge may have been affected by changed watershed conditions. This peak is much greater than the 100-year peak discharge estimate from either *HEC-1* or *FLO-2D*. The *FCDMC* web site also lists a table of flood frequency estimates as shown in Table 4.47. That table lists the 100-year peak discharge as 6,900 cfs, which is much closer to the peak from the *FLO-2D* model. The web site flood frequency estimates are from a multiple sub-basin *HEC-1* model. Subdividing a watershed into multiple smaller sub-watersheds will usually result in an increased peak discharge.

Table 4.46 Gage 4963 Measured Peak Discharges by Water Year

Water Year	Peak Gage Height (feet)	Peak Discharge (cfs)	Date of Peak
2015	None	0	None
2014	5.42	1,089	08/19/2014
2013	2.35	125	03/08/2013
2012	6.60	1,843	09/07/2012
2011	1.72	44	09/10/2011
2010	8.26	2,985	01/21/2010
2009	3.50	331	02/09/2009
2008	5.30	1,021	12/01/2007
2007	2.88	214	10/14/2006
2006	7.38	2,464	07/30/2006
2005	10.68	11,700	09/03/2005
2004	0.93	27	11/13/2003
2003	1.25	42	02/26/2003
2002	0.80	22	09/07/2002

Table 4.47 Gage 4963 Flood Frequency *HEC-1* Data from *FCDMC* Web Site

Flood Flow Frequency - HEC-1 Analysis of June, 2002					
Magnitude and Probability of Instantaneous Peak Flow | Discharge, in cfs, for indicated Recurrence Interval					
NOTE: Flood Flow Frequency data are for information only and should not be considered valid for regulation					
2-year	5-year	10-year	25-year	50-year	100-year
1,390	2,760	3,540	4,740	5,850	6,900

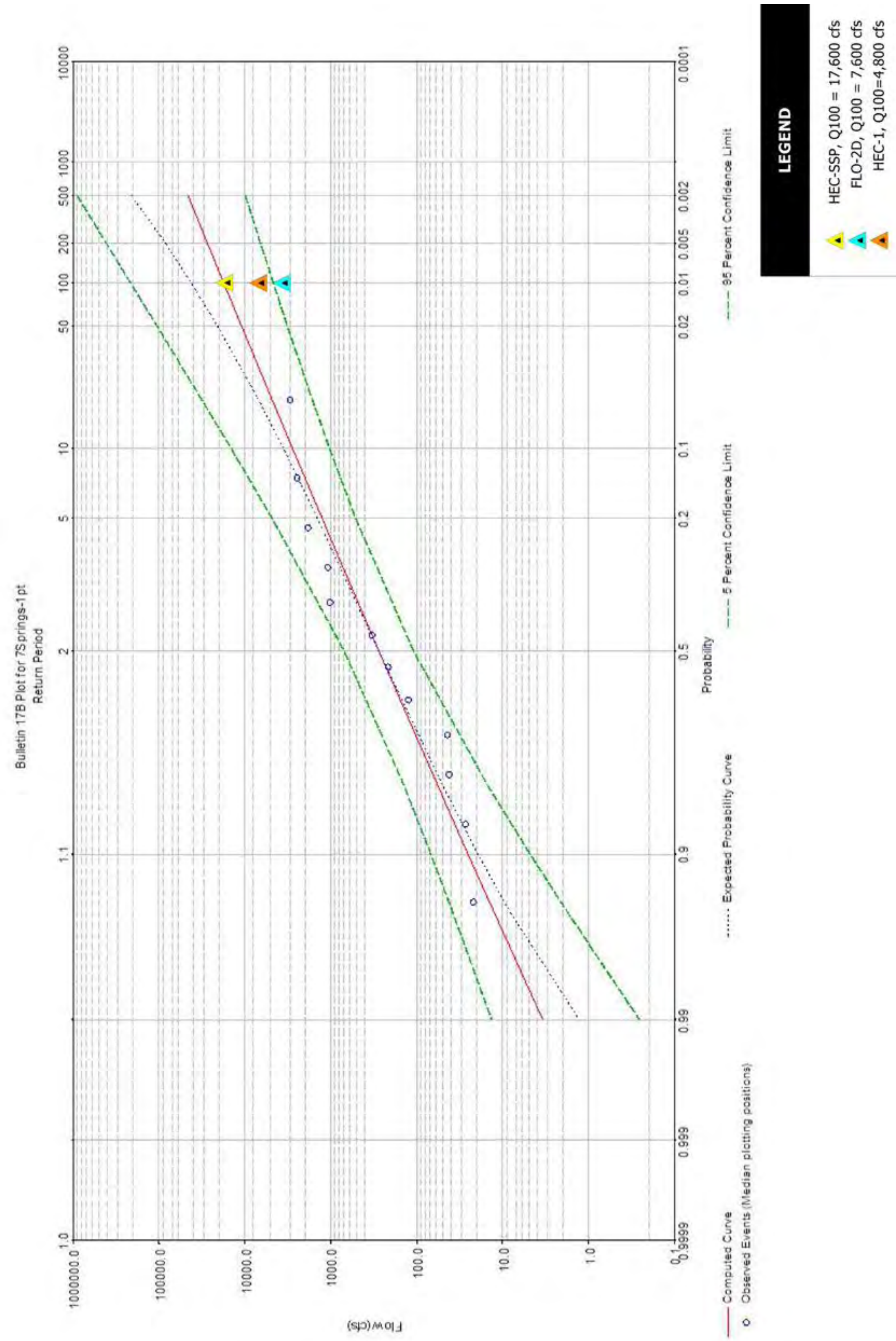
The annual peak discharge values from Table 4.46 were used to perform a flood frequency analysis for the gage location. The *HEC-SSP* (USACE, 2010b) program version 2.0 was used for this analysis. The Bulletin 17B procedures were applied. The results are shown on Figure 4.300. This analysis treated the 2005 peak as an outlier. The HEC-SSP 100-year estimate is 17,600 cfs. The *HEC-1* 100-year discharge falls below the 5% confidence limit, and the *FLO-2D* 100-year discharge above the 5% confidence limit. Even with the limited number of years of record, the *FLO-2D* estimated 100-year discharge is not unreasonable and may actually be low.

Discussion

Modeling known storm events with a hydrologic model is always problematic. As described previously for the Rainbow Wash test cases, there are many variables with a fairly high degree of uncertainty for each. With these uncertainties in mind, the Seven Springs Wash *FLO-2D* storm replication models do a very good job of reproducing the measured hydrographs. Both test cases provide better results than the *FCDMC*'s current *1D* modeling method. The success of modeling the two actual storms provides confidence that the *FLO-2D* model will provide more accurate results for this watershed when a design storm is applied.

The standard *FCDMC* 100-year 24-hour design storm was applied using the *FLO-2D* model and the results compared with *HEC-1,* the *FCDMC* Indirect Methods confidence checks, and a flood frequency analysis of the Gage 4963 annual peak discharge history. The *FLO-2D* hydrograph has a reasonable shape and volume and the peak discharge reasonably compares with the majority of 100-year peak discharge estimates from gaged watersheds with similar area and topography. The *FLO-2D* result also fits within the confidence limits from the flood frequency analysis and is less than the historical peak discharge of record. Based on these checks, the *FLO-2D* model is capable of reasonably representing actual flood events and applying the *FCDMC* 100-year 24-hour design storm.

Figure 4.300 Flood Frequency Analysis Results for Gage 4963



4.3.1.4 Guadalupe FRS Test Case

General

The Pima Canyon Wash in Maricopa County was selected for a storm reconstitution test case. The location of the Pima Canyon Wash watershed is shown on Figure 4.270. The watershed for Pima Canyon Wash at the Guadalupe FRS is shown on Figure 4.301. The watershed area is approximately 1.84 square miles. *FCDMC* owns and maintains a recording pressure tranducer stage gage/rain gage (*FCDMC* Gage 6500), which has been in operation since June 29, 1989. Refer to the Guadalupe FRS Vicinity Map shown on Figure 4.302. The *FCDMC* also owns and operates a rain gage in the watershed: Gage 6510 South Mountain Park, as shown on Figure 4.301. The wash is a well-defined channel with a typically trapezoidal cross section, a bed of sand, gravel, and large cobbles underlain by rock outcrop and caliche in various locations, and well vegetated banks. The majority of Pima Canyon Wash is a natural channel. In the lower portion of the watershed it is a constructed urban channel with limited capacity. Refer to photographs of the wash shown on Figure 4.303. Only one major runoff event has been recorded at Gage 6500 between 1989 and 2008 that resulted in a percent storage greater than 10% of capacity, July 13, 2008.

Figure 4.301 Pima Canyon Wash Watershed Map

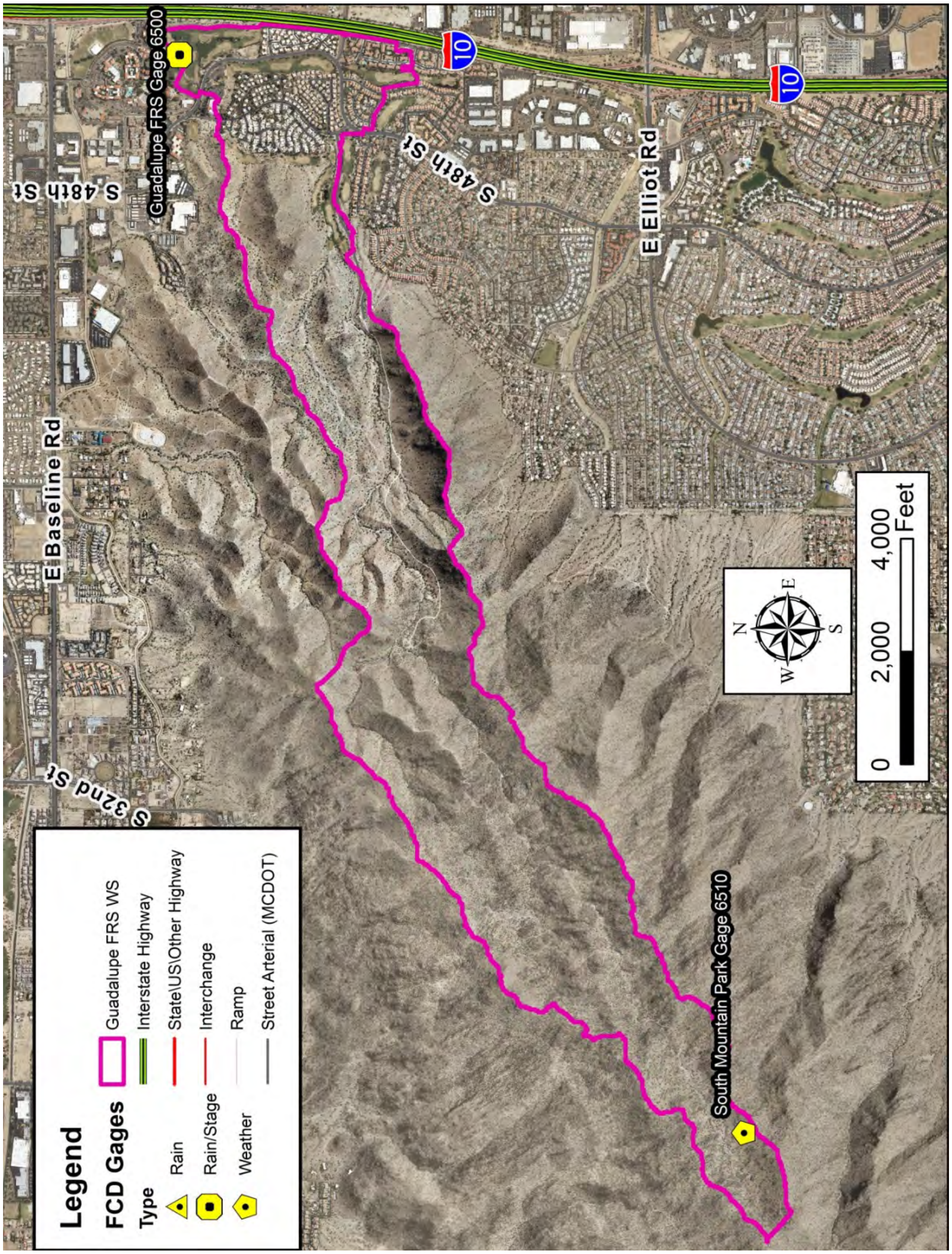


Figure 4.302 Guadalupe FRS Vicinity Map

Figure 4.303 FCDMC Guadalupe FRS Watershed Photographs

Typical channel in upper natural watershed

Typical channel in lower natural watershed

Typical channel in urban area

FRS post-storm 7/13/2008

The *FLO-2D* model used to simulate this storm was developed in-house for the purposes of this study. The model development and parameters are documented in *Guadalupe FRS Flood Retarding Structure Watershed Hydrology and Dam Breach Analysis* (FCDMC, 2015b). The key information for the model of the Pima Canyon Wash Watershed includes:

- FLO-2D Pro Build 2009.18.03
- 25-foot grid size. The model is intended for hydrology, not detailed hydraulics.
- Topography based on the *FCDMC* 10-foot contour interval digital terrain model, flight date 2001 for South Mountain, and a combination of 2-foot contour mapping and field survey described in *FCDMC* (2015b).
- *G&A* rainfall loss method using *FCDMC* standard parameters and Saxton, et al., 2006.

Storm *GARR* data was generated by METSTAT, Inc. for the storm as briefly described in Section 4.3. Refer to the following report for documentation of the generation of the GARR data:

8. *Storm Precipitation Analysis Report, Storm of July 13-14, 2008, Maricopa County, Arizona SPAS Storm #1240* (METSTAT, Inc., 2012e).

The *GARR* data was used to create the *FLO-2D* RAINCELL.DAT input data file, which simulates the actual moving storm rainfall using 5-minute time intervals. The *FLO-2D* model routes the watershed runoff into the Guadalupe *FRS* and simulates the storage ponding within the impoundment. The resulting stage versus time hydrograph from the *FLO-2D* model at the grid element containing *FCDMC* Gage 6500 was then compared with the measured stage versus time data from the gage. The results are presented and discussed in the following section.

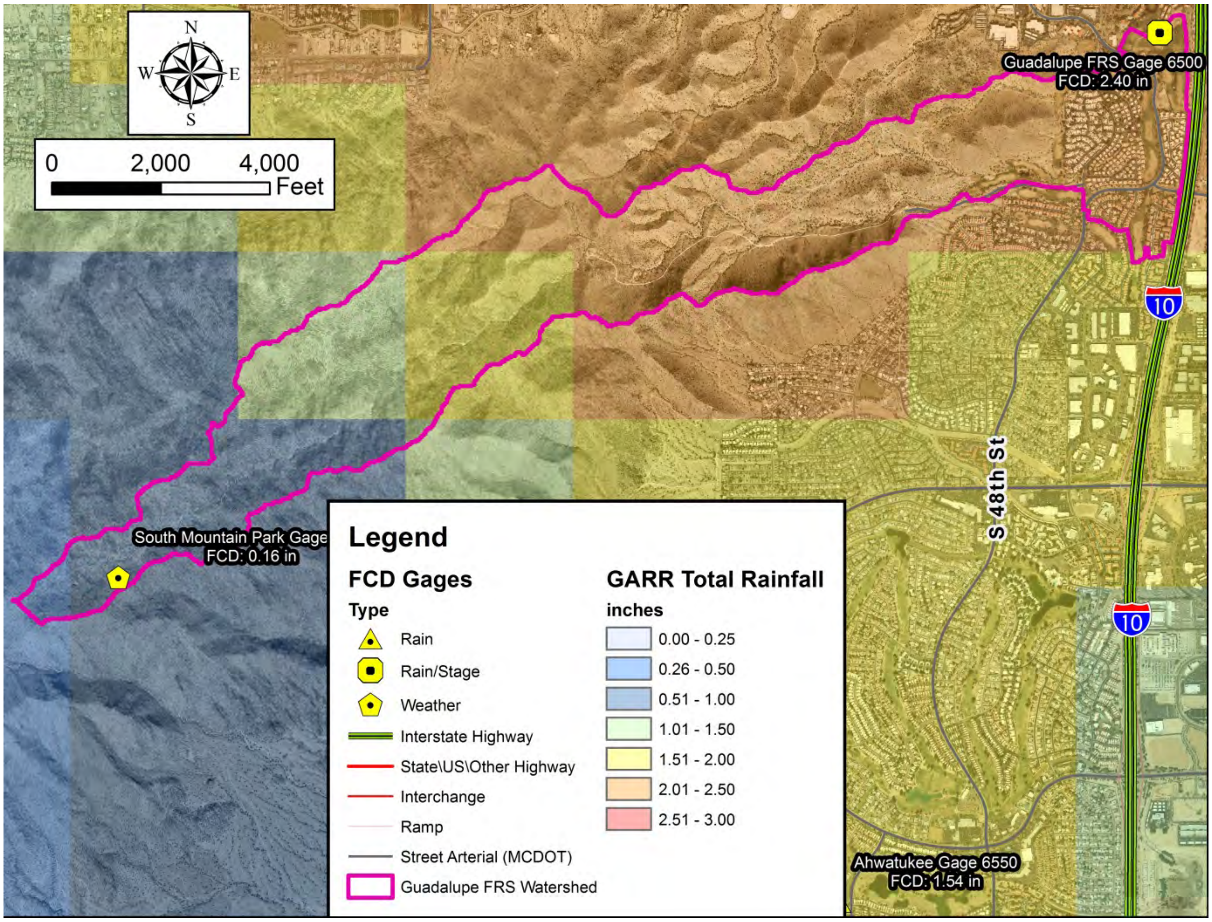
Storm of July 13, 2008

A significant storm event occurred July 13, 2008 that resulted in impounding of water within the Guadalupe *FRS*. This was the most significant event to occur at the structure since the gage was installed in 1989. The total storm rainfall values from the *GARR* data are shown on Figure 4.304. The locations of the three local rain gages and the Guadalupe *FRS* stage gage are also shown. This represents the rainfall data used in the *FLO-2D* models of the storm. Note that the storm produced significantly greater rainfall totals over the eastern half of the watershed than on the west end near the top of South Mountain.

There were two observer gages in the immediate area of Gage 6500 that provided total point rainfall values for the storm. Both are shown on Figure 4.304. Note that the total point rainfall values at these gages are an indication that there may have been much higher rainfall at isolated locations in the urbanized area closer to the *FRS* than measured at the official *FCDMC* gages or as indicated by the *GARR* data. *GARR* data values at specific points where there is no measured data are shown with a brown filled triangle labeled "*GARR*" on Figure 4.304. The two observer network values were not used in the preparation of the *GARR* data. However, the *GARR* data estimates higher rainfall values than measured at the *FCDMC* gages when all the *FCDMC* gages within a 25 km radius are taken into account. The increase in runoff volume that could result from the overestimation of rainfall is as much as 10 ac-ft (*FCDMC*, 2015b).

Results from the two *FLO-2D* models are shown on Figure 4.305. The *WSEL* (Stage) in the *FRS* at Gage 6500 is shown over time for both *FLO-2D* models and the those measured at the gage during the storm event. Also shown is the rainfall measured at Gage 6500.

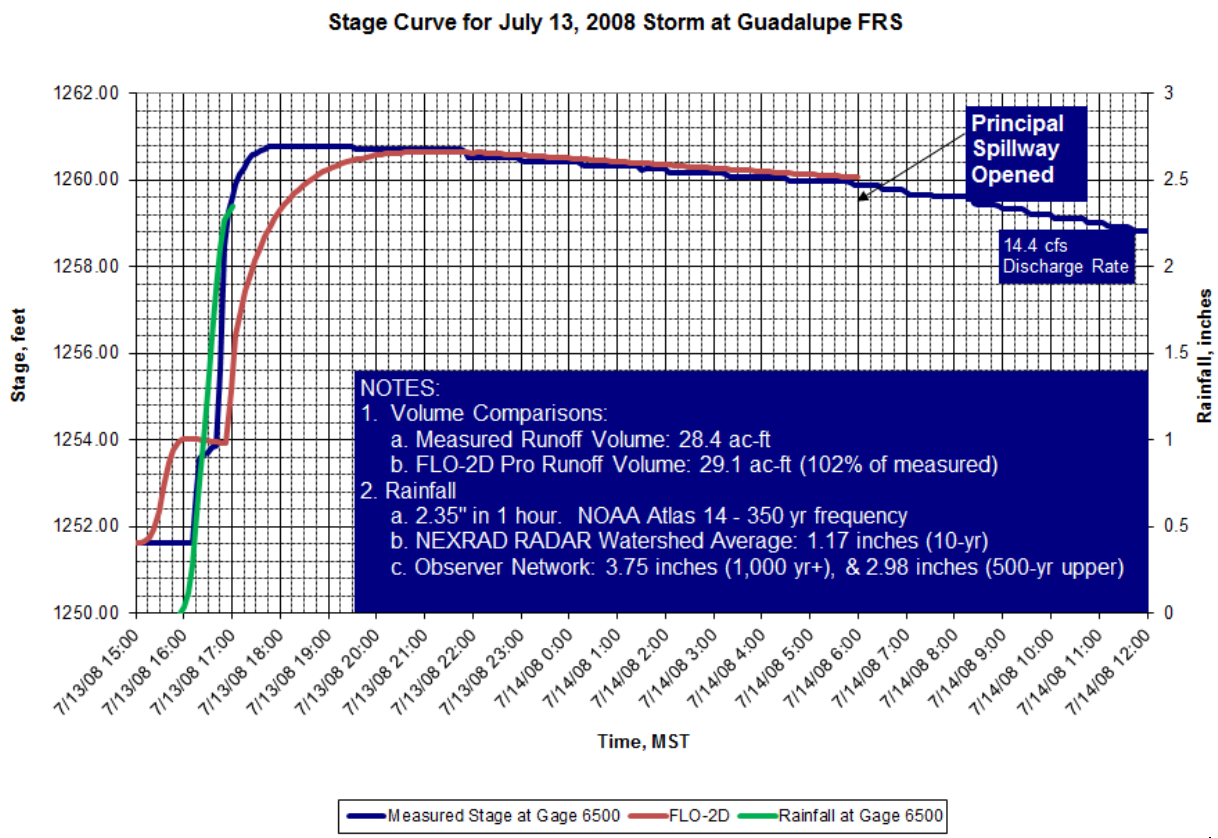
Figure 4.304 July 13, 2008 Storm GARR Total Rainfall and Gage Locations



Discussion

Note from Figure 4.305 that the rising limb of the *FLO-2D* model stage hydrograph begins much sooner than measured but that a longer delay in response also occurs between 15:45 and 17:00 hours. The rising limb of the *FLO-2D* model results is also not as steep as measured. This is probably due to intense rainfall over the urban area adjacent to the *FRS* that is not reflected in the *GARR* data, as witnessed by the observer gages. The *FLO-2D* model yielded nearly identical total runoff volume to what was measured at Gage 6500. The *FLO-2D* model does a good job representing this storm. Based on these tests, the *FLO-2D* model is capable of reasonably representing actual flood events.

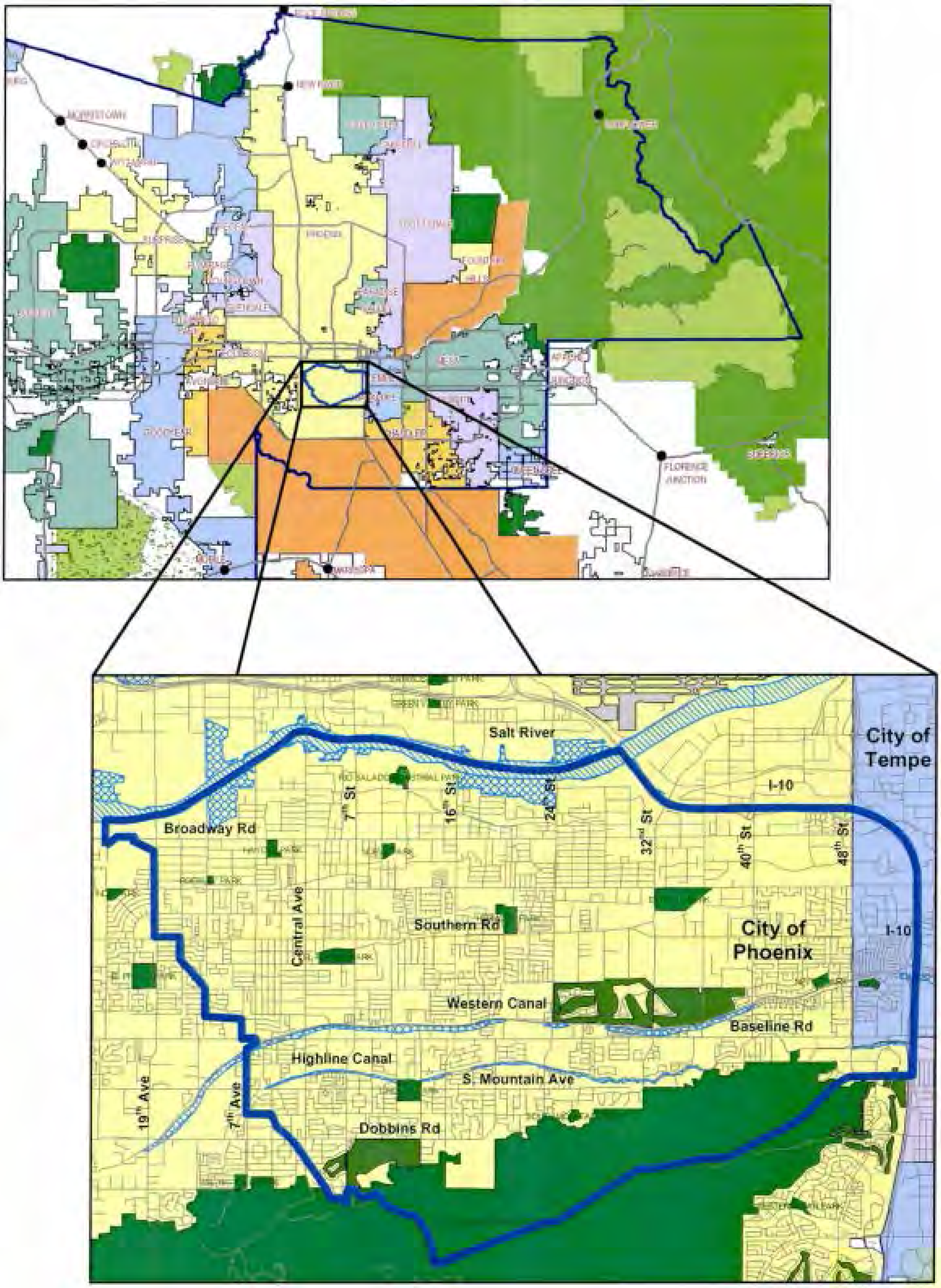
Figure 4.305 FLO-2D Results Compared with Measured at Gage 6500



4.3.1.5 Hohokam ADMS Test Cases

The Hohokam *ADMS* was completed in April 2012 by Stanley Consultants, Inc. under *FCDMC* Contract FCD 2009C029. Refer to Figure 4.306 for a map of the project study area in southeast Phoenix. The Hohokam Area Drainage Master Study/Plan (Stanley Consultants, Inc., 2012) is a two-phase regional flood control planning project to determine the nature and magnitude of existing flood hazards; develop and evaluate potential mitigation alternatives; provide preliminary design plans for recommended improvements; and ultimately to provide a comprehensive plan to address flooding within the study area and guide future development and flood control improvements. As a part of the *ADMS* phase, several *FLO-2D* models were developed to determine the nature and magnitude of existing flood hazards.

Figure 4.306 Hohokam ADMS Project Location and Vicinity Map



The models, which cover the entire watershed, were based on:

- *FLO-2D* 2009.06 Build No: 09-11.07.06 (64-bit),
- 30-foot grid size,
- included infiltration and building and wall obstructions,
- very detailed depiction of impervious area, storm drains and culverts modeled as hydraulic structures, and
- failure of walls based on structural characteristics and ponded flood depths.

Two storm events occurred in the study area (July 13, 2008 and July 31, 2010) during the *ADMS* project development and provided the study consultant team an opportunity to perform confidence checks on the results of the *FLO-2D* analyses. For both events, *GARR* data was obtained and the storm events simulated in *FLO-2D*. Ideally, it would be best to compare simulation results to quantifiable data collected in the field from monitored or gauged conveyances or basins. Lacking such quantitative data, simulation results were qualitatively compared to field observations (e.g. high water marks), drainage complaints, eyewitness accounts, and available photographic evidence.

Storm of July 13, 2008

The July 13, 2008 storm resulted in isolated flooding primarily in the eastern portion of the study area. It is generally known that this event resulted in flooding in western Tempe and resulted in the closure of a portion of US 60 due to the failure of a freeway pumping station. Recorded drainage complaints and collected residential photos specifically document the failure of a residential wall and flooding in the Cortland Point Subdivision (32nd St south of Baseline Road), the parking lot of Shadow Mountain Condos (36th St and Baseline) and the flooding of an industrial building in the vicinity of 46th St and Beautiful Lane. The total rainfall for this event occurred in roughly a 1 hour period and is depicted on Figure 4.307. Also shown are the *FCDMC* rain gage locations and the locations used for model verification.

Storm of July 31, 2010

The total rainfall for this event occurred in roughly a 1 hour period and is depicted on Figure 4.308. Also shown are the *FCDMC* rain gage locations and the locations used for model verification. On July 31, 2010, a significant monsoon storm hit the study area resulting in flooded roadways, canals, homes and properties in isolated locations. Rainfall and flooding was most intense in the south central portion of the study area where more than an inch of rainfall fell in less than one hour. Several properties suffered damage from flooding, erosion and sediment deposition.

Figure 4.307 Total Rainfall for Storm of July 8, 2008

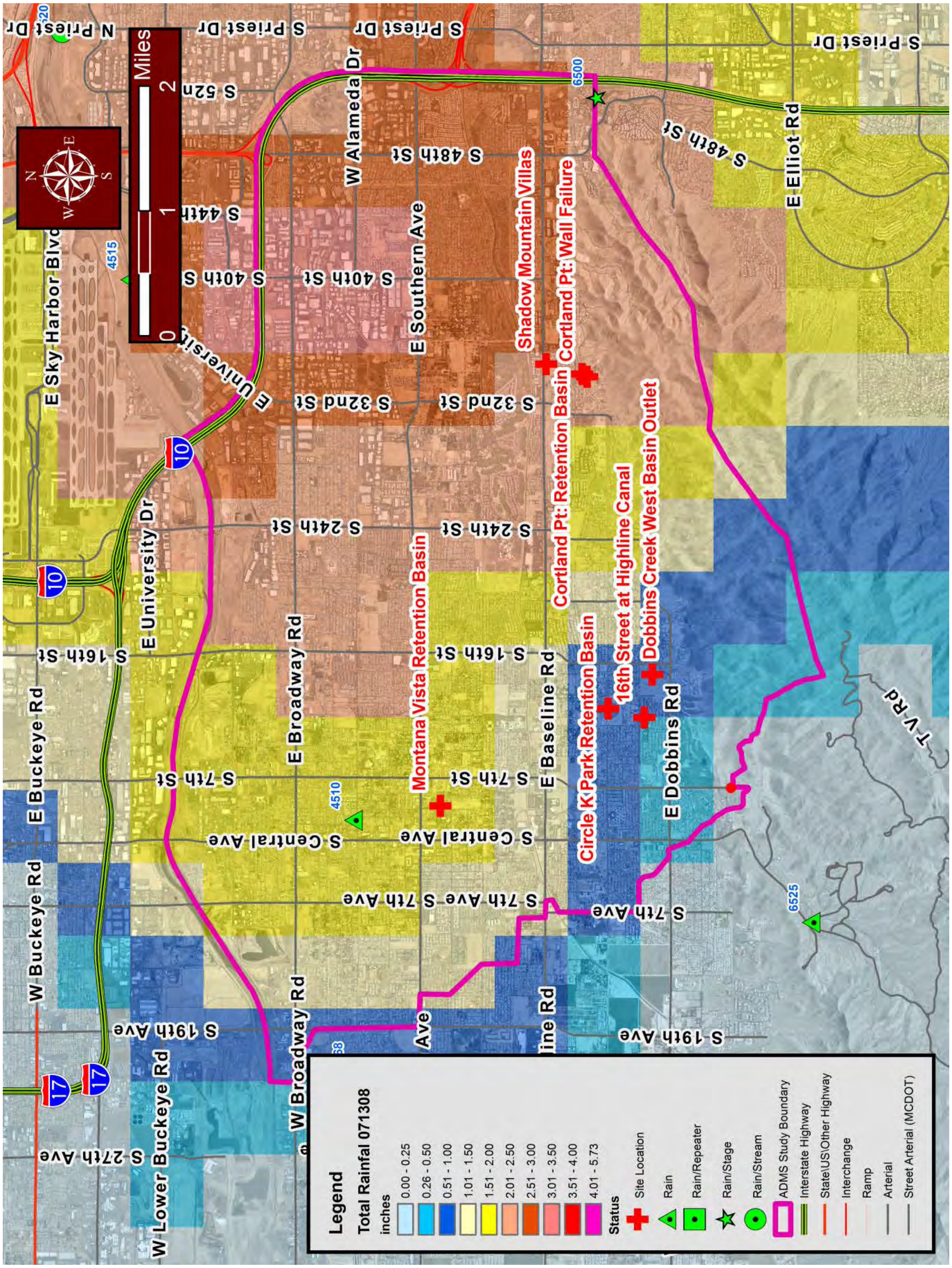
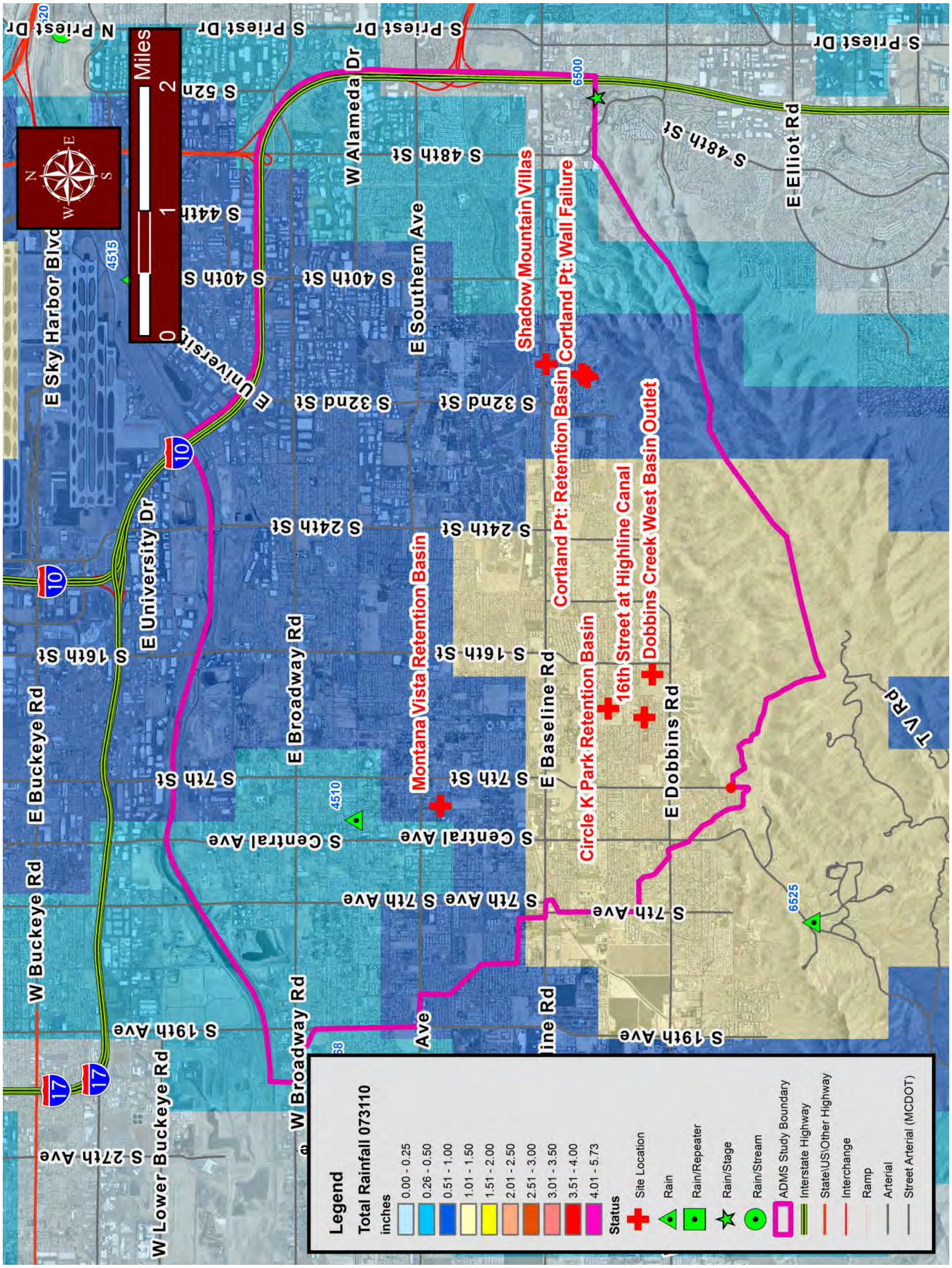


Figure 4.308 Total Rainfall for Storm of July 31, 2010



Most of the flooding complaints and reports were documented south of, and along, the Highline Canal between 7th Street and 28th Street. However, significant flooding also occurred further downstream (north) of the Highline Canal at 13th Place (just north of Circle K Park), 16th Street at 21st Street (Pines at South Mountain development). In addition to flooding, a significant amount of sediment and debris was deposited in the streets, catch basins, retention basins and swimming pools presenting a maintenance headache for residents and City of Phoenix maintenance personnel.

For the July 31, 2010 event, several problem areas were identified that were instrumental for the consulting team in evaluating *FLO-2D* results both in flooding location and magnitude. These were:

- Overtopping of Highline Canal at 14th St
- Flooding along 16th St, specifically at 16th St and the Highline Canal
- Dobbins Creek Retention Basins
- Montana Vista Retention Basin
- Flooding along the Highline Canal, at the Pines at South Mountain Development
- Cortland Point Subdivision

Discussion

The comparisons between the *FLO-2D* storm model results and field observations and photograph documentation are shown in Table 4.48 (Stanley Consultants, Inc., 2012). There is reasonably good correlation between the *FLO-2D* model results at all locations. The results for the 100-year 6-hour storm

FLO-2D model based on the *FCDMC* design storm are shown for comparison. That model is based on existing conditions at the time the Hohokam ADMS was prepared. The results for these three storms demonstrate that *FLO-2D* can reproduce actual storm results very well, which means that both the hydrology and hydraulics of those events are replicated to a reasonable level of accuracy.

Table 4.48 Hohokam *ADMS FLO-2D* Verification Comparisons

Location	Field Observations and Photo Documentation	FLO-2D GRID ID	Model Runs		
			July 13 2008	July 31 2010	Exist 100-yr, 6-hr
Montana Vista Retention Basin	7/31/10: 4.0 ft	118448	-	3.96 ft	7.66 ft
Dobbins Creek West Basin Outlet	7/31/10: 3.0 ft	129823	-	1.67 ft	6.85 ft
Circle K Park Retention Basin	7/31/2010: ~ 0.5 ft across Highline Canal (~ 20 cfs thru three shallow culverts)	202062	-	0.48 ft (54 cfs)	1.21 ft (376 cfs)
16th Street at Highline Canal	7/31/10: 0.5 ft in street	213810	-	0.38 ft	0.78 ft
The Pines at South Mountain	7/31/10: NW retention basin was full to capacity, 3.0 ft	281241	-	3.26 ft	4.75 ft
Cortland Pt: Wall Failure on Francisco Drive	7/13/08: ~2 ft	220120	2.30 ft	-	2.33 ft
Cortland Pt: Retention Basin on Francisco Dr	7/13/08: 2.5 ft in retention basin, street ponded, overflow to north along 25th St	227198	2.27 ft	-	2.55 ft
Shadow Mountain Villas (parking lot)	7/13/08: 4 ft in parking lot along Baseline Rd	-	3.4 ft		

4.3.1.6 Beaver Dam Wash Test Case

General

A major long-duration winter storm occurred over the Beaver Dam Wash watershed December 17, 2010 through December 23, 2010. The 576 sq-mi watershed lies in Arizona, Nevada and Utah as shown on the location map in Figure 4.309. A more detailed map of the watershed is shown on Figure 4.310. The small community of Beaver Dam in Arizona experienced severe flooding as a result of this storm. Four (4) homes in Beaver Dam Resort were totally destroyed and two (2) others extensively damaged. Lateral migration of the southwest bank destroyed the homes and removed a portion of Clark Gable Drive and a side street and removed the wastewater lift station.

Storm of December 17-23, 2010

This storm was modeled by Mohave County Flood Control District for the purpose of preparing a flood warning response plan for the Beaver Dam area. The information presented herein is taken from the *Beaver Dam, AZ Flood Warning Response Plan Hydrology and Hydraulics Report* (MCFCD, 2014). As a part of the study, the Beaver Dam Wash through the community of Beaver Dam was modeled using the following:

- *FLO-2D* Pro Build 13.02.04, and
- A 15-foot grid size.

The estimated flood hydrograph from the 2010 event (MCFCD, 2014) was routed through the model. The peak discharge was estimated to be about 13,300 cfs with a frequency estimate of about 30-years. The storm hydrograph was routed through the model to verify that the *FLO-2D* model of Beaver Dam Wash produced reasonable results when compared with the observed. The model was calibrated through small adjustments in roughness and used in development of the flood warning response plan.

The 2010 flood limits were defined using February 2011 aerial photographs prepared by Cooper Aerial Mapping. The limits are based on visible flood marks and field reconnaissance (MCFCD, 2014). The estimated flood limits overlaid on the 2011 post-flood aerial photograph are shown on Figure 4.311. The estimated flood limits compared with the *FLO-2D* model results are shown on Figure 4.312. The *FLO-2D* model replicates the visible flood limits very well.

Discussion

The results of this test show that *FLO-2D* can reproduce an actual storm event with reasonable accuracy and minimal parameter adjustments for calibration against know flooding extents.

Figure 4.309 Beaver Dam Wash Watershed Location Map

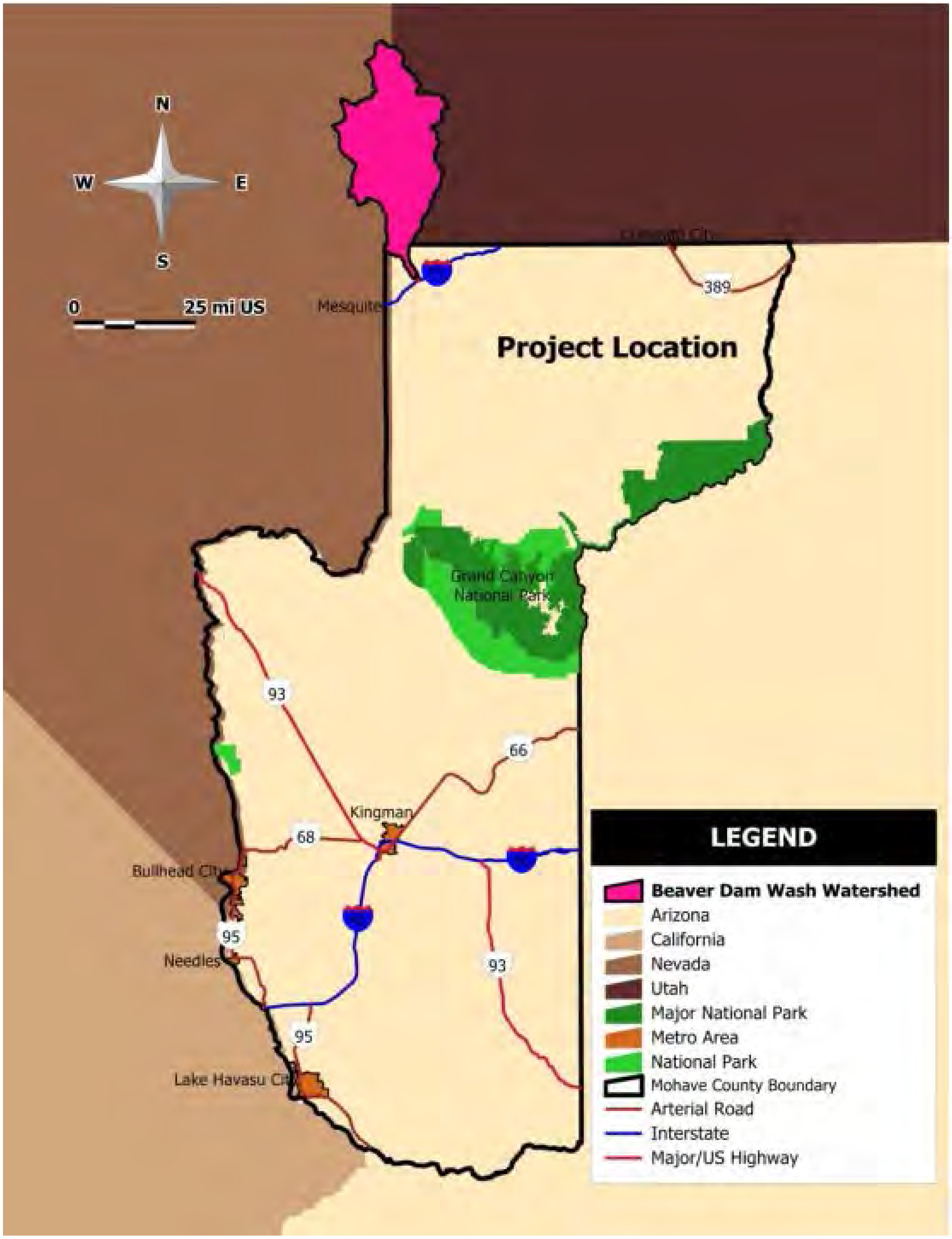


Figure 4.310 Beaver Dam Wash Watershed Map

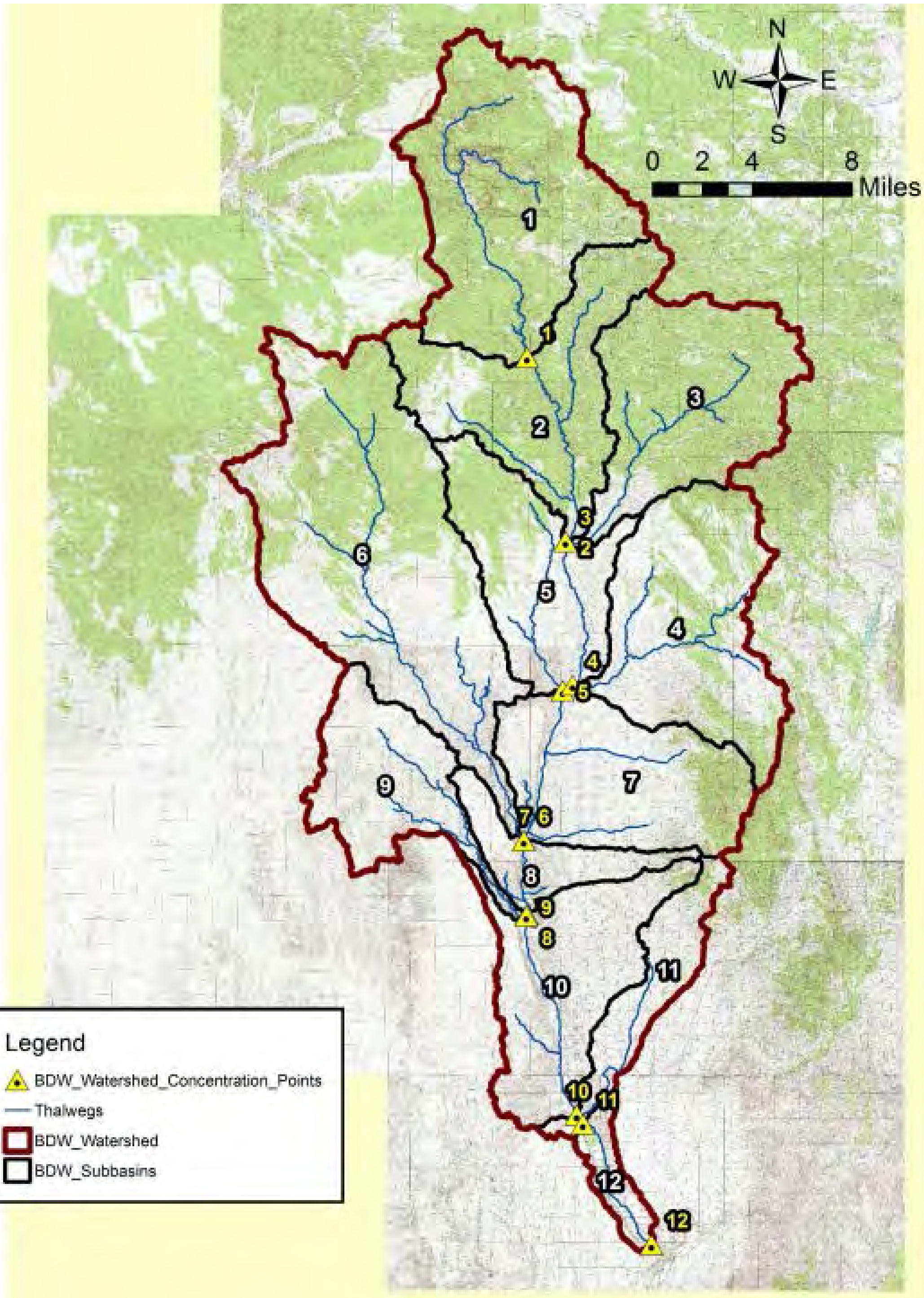
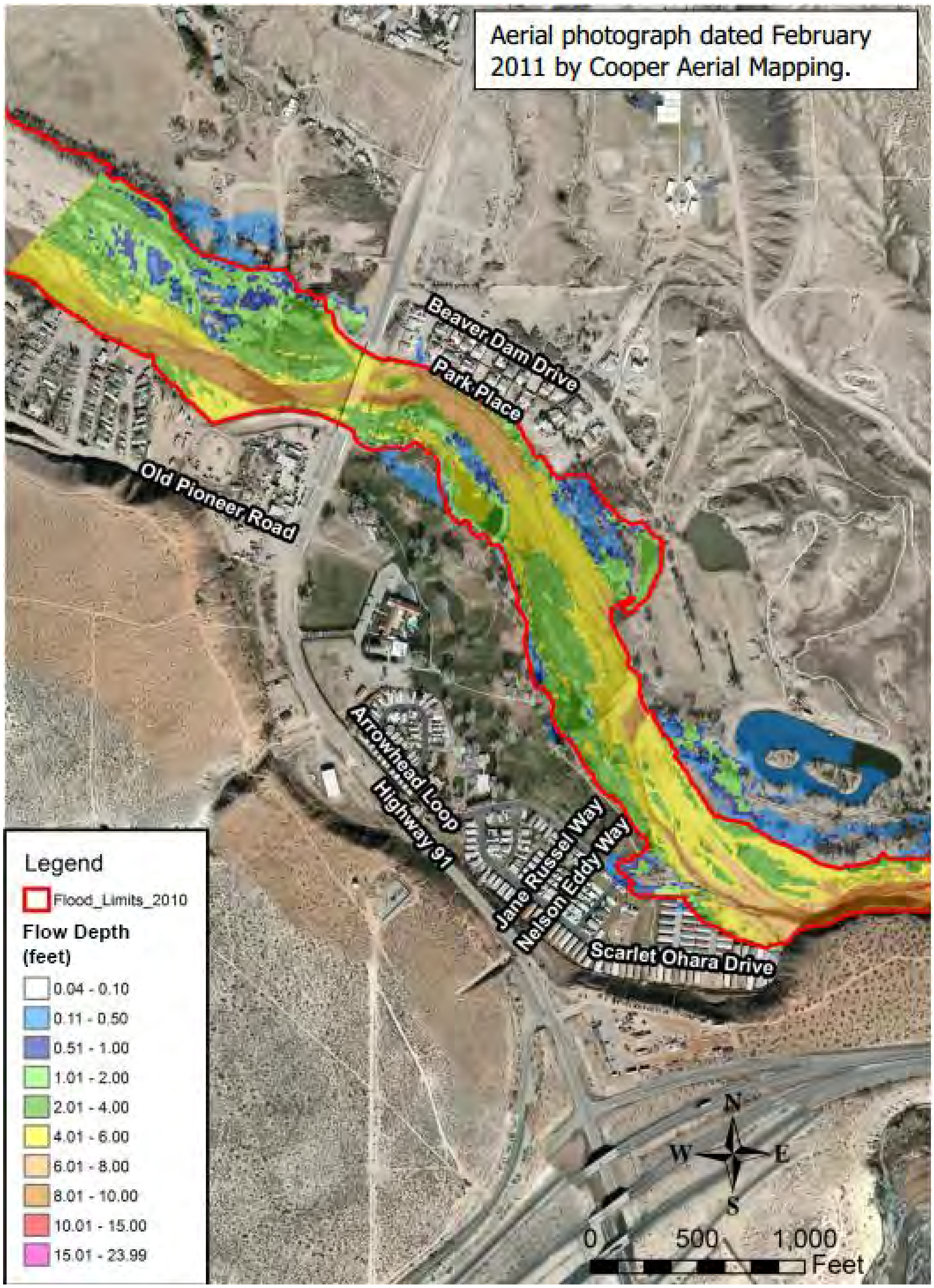


Figure 4.311 Beaver Dam Wash at Beaver Dam, AZ Estimated Flood Limits

Figure 4.312 Beaver Dam Wash at Beaver Dam, AZ *FLO-2D* Model Flood Limits



4.4 Suitability for Delineation of Floodplain Limits

Two of the storm reconstitution test cases presented herein shows that *FLO-2D* can accurately reproduce flood limits from actual storms. Refer to the storm of August 9, 2005 on Rainbow Wash (Section 4.3.1.2) and the storm of December 17-23, 2010 on Beaver Dam Wash (Section 4.3.1.6).

The Rio Verde *ADMP* prepared by the *FCDMC* in 2007 using *FLO-2D* 2009.06 is an example of a floodplain delineation prepared using 25-foot grid model and using the grid boundaries to define the floodplain limits ((FCDMC, 2007). Refer to Figure 4.313 for the AE Zone floodplain areas approved by *FEMA* based on the *FLO-2D* model results. As a part of the approval, a *FEMA* Floodplain Boundary Standard Check was performed. This check justifies using the grid boundaries without smoothing, which was accepted by *FEMA*.

The procedure followed was to create a surface using the maximum water surface elevation computed for each grid element as the Base Flood Elevations for the delineation. Base flood water surface elevation contour lines were created using this surface, and then each base flood contour line was intersected with the floodplain boundary where a point was set in a *GIS* feature class. The ground elevation at each of these points was determined using from the existing ground surface *DEM* used for the hydraulic modeling. The ground surface elevation at each point was compared with the base flood water surface elevation (*BFE*). The check resulted in a total of 7066 points analyzed.

The Rio Verde area best fits into Risk Class B with some potential for Risk Class A in the future. Refer to Table 4.49. The analysis resulted in a 97.2% of points passing, meeting the Risk Class A requirements.

FCDMC prefers to use the grid boundary for the floodplain limit line in unconfined flow areas, but smoothed boundaries can also be generated particularly for confined flow riverine conditions.

Figure 4.313 Rio Verde ADMP 100-year Floodplain Delineation Map

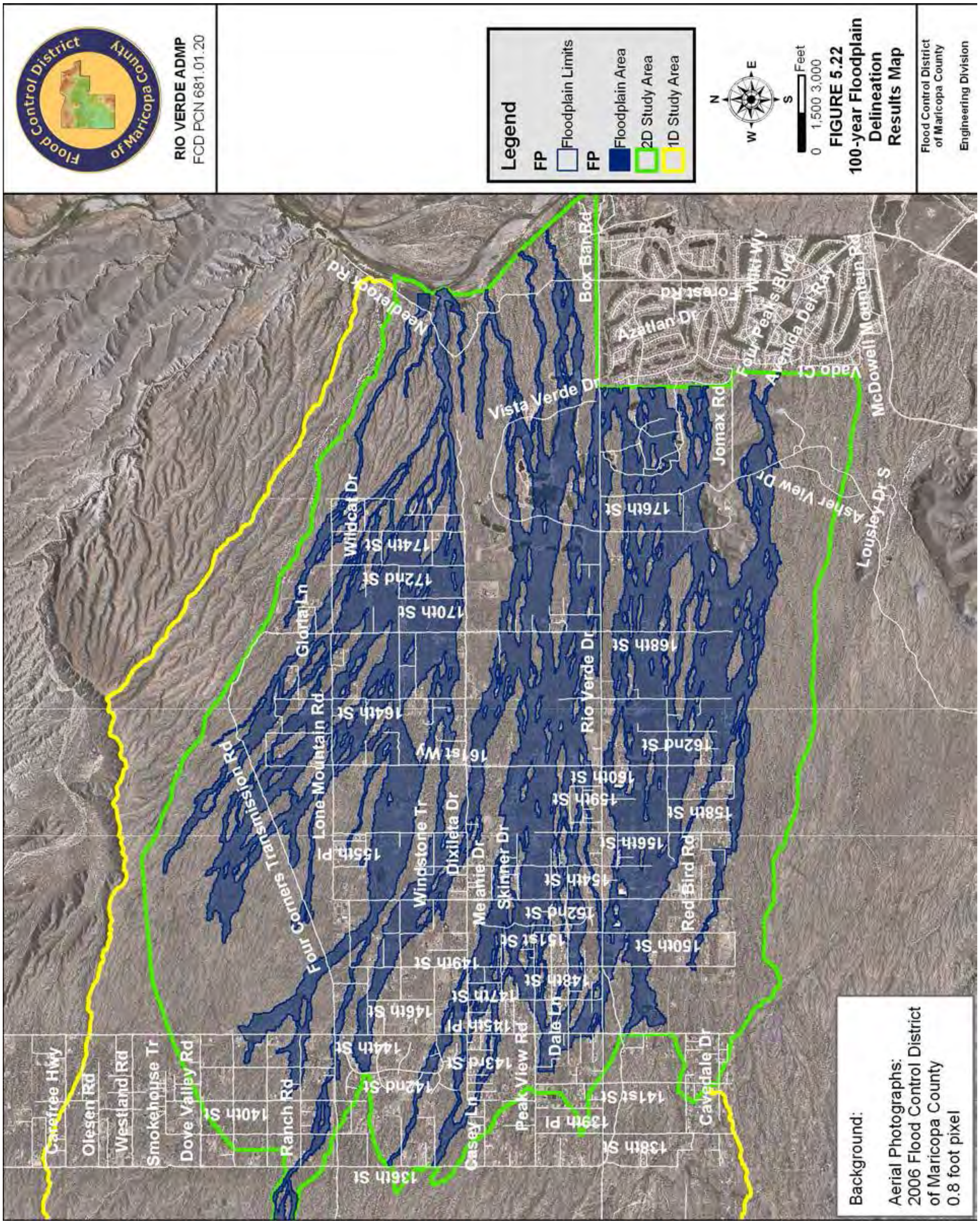


Table 4.49 *FEMA* Floodplain Boundary Standard for Flood Insurance Rate Maps

Table 1. Floodplain Boundary Standard for Flood Insurance Rate Maps

		Delineation Reliability of the floodplain per study methodology [1]	
Risk Class	Characteristics	Detailed	Approximate [2]
A	High population and densities within the floodplain, and/or high anticipated growth	+/- 1.0 foot/ 95%	+/- 1/2 contour 95%
B	Medium population and densities within the floodplain, and/or modest anticipated growth	+/- 1.0 foot/ 90%	+/- 1/2 contour 90%
C	Low population and densities within the floodplain, small or no anticipated growth	+/- 1.0 foot/ 85%	+/- 1/2 contour 85%
D	Undetermined Risk, likely subject to flooding	NA	NA
E	Minimal risk of flooding; area not studied	NA	NA

4.5 Summary and Conclusions

The verification tests performed and documented in Section 4 show that *FLO-2D* 2009.06 and *FLO-2D* Pro accurately perform hydrologic and hydraulic calculations when compared with industry standard software applications prepared and supported by the Federal government. The tests also show that the program meets *FCDMC* technical standards set forth in the *DDM Hydrology* (FCDMC, 2013a) and *DDM Hydraulics* (FCDMC, 2013b).

5 REPORTING CAPABILITIES

5.1 Purpose

There are several methods used by *FLO-2D* to report results and these include hydrographs, maps, profiles and animations. The purpose of this section is to summarize these methods and tools, but not to provide detailed instructions in their application. Instructions and examples are included in the *FLO-2D* manuals and workshop lessons. The exception is the SUMMARY.OUT file; therefore, a detailed description is covered in Section 5.2.

5.2 SUMMARY.OUT

5.2.1 General

The SUMMARY.OUT file has a significant amount of critical information pertaining to the performance of the model and should be carefully reviewed. The SUMMARY.OUT file contains the *FLO-2D* build number of the executable used, followed by various statistics and model output summary data, with the computation run time and the date and time the model run was completed at the end of the file. The following sections will guide the user through the important summary output data.

5.2.2 Volume Conservation Table

The *FLO-2D* SUMMARY.OUT file reports the volume conservation by output time interval (TOUT typically in hours). Data in the table columns include:

1. Reporting time interval
2. Average computational time step during the reporting time interval
3. Volume conservation error in acre-feet (or cubic meters)
4. Volume conservation error in percent of total inflow

5.2.3 Mass Balance: Inflow – Outflow Volume

5.2.3.1 Inflow (acre feet)

An example of an Inflow summary section from a *FLO-2D* model created as a part of the Tempe *ADMS* is shown on Figure 5.1. Each summary statistic is described below.

TOTAL POINT RAINFALL

The total point rainfall (TPR) of 3.4400 inches is applied before any adjustments such as a depth-area factor. This point rainfall is reported as the sum of the incremental time step uniform point rainfall values applied to every grid element without depth variable adjustment.

RAINFALL VOLUME

The value of 2657.71 acre-feet is the total rainfall volume applied to the entire model domain. Rainfall is not applied to outflow grids and the total rainfall depth applied to any individual grid may be adjusted by the spatially-assigned variable RAIN*ARF*(I) in the RAIN.DAT input data file. The total rainfall volume (TRV) can be verified as follows, assuming RAIN*ARF*(I) values have not been assigned, which is the case for this example:

1. Total number of grids (NG): 1,010,360
2. Grid size: 20 feet
3. Grid area (GA): 400 sq-ft
4. Total number of outflow grids (NOG): 742
5. Rainfall Area (RA)
6. RA = (NG-NOG)*GA/43,560 sq-ft/ac = (1,010,360-742)*400/43,560 = 9,271.056 acres
7. Average point rainfall (APR): 3.4400 inches (for this case, APR is equal to TPR)
8. TRV = APR/12*RA = 3.44/12*9,271.056 = 2,657.70 ac-ft

The computed TRV checks against the value reported in SUMMARY.OUT. If RAIN*ARF*(I) values are assigned in RAIN.DAT, then the APR must be calculated by multiplying the RAIN*ARF*(I) for every cell by TPR. If RAIN*ARF*(I) values are not assigned for some cells, then the TPR should be used for those cells. If RAIN*ARF*(I) value assignments are zero then those grids should be removed and also not included in RA. The resulting point rainfall value for every cell within the RA should then be summed to obtain TRV.

SURFACE WATER INFLOW HYDROGRAPH

The value of 766.68 ac-ft can be verified using the INFLOW.DAT file and computing the inflow volume from each inflow hydrograph using a spreadsheet.

INFLOW HYDROGRAPHS + RAINFALL (TIV)

This value (3,424.40 ac-ft) represents the total inflow volume imposed on the model domain, including rainfall and inflow hydrographs.

Figure 5.1 SUMMARY.OUT: Inflow Section

```
================================================================================
                  MASS BALANCE    INFLOW - OUTFLOW VOLUME
================================================================================
                         *** INFLOW (ACRE-FEET) ***

TOTAL POINT RAINFALL:                                         3.4400 INCHES


                                                                 WATER

RAINFALL VOLUME                                                2657.71

SURFACE WATER INFLOW HYDROGRAPH                                 766.68
                                                             ---------
INFLOW HYDROGRAPHS + RAINFALL                                  3424.40
```

5.2.3.2 Surface Outflow

An example of a Surface Outflow summary section from a *FLO-2D* model created as a part of the Tempe *ADMS* is shown on Figure 5.2. Each summary statistic is described individually below.

WATER LOST TO INFILTRATION & INTERCEPTION (WLII)

This value (651.05 ac-ft) is described first because information from its verification is needed to check the OVERLAND INFILTRATED AND INTERCEPTED WATER value. The WLII value is the total volume of infiltrated water lost to the pervious area (PA) plus the volume of water lost to initial abstraction for all grid elements in the RA. It is important to note that infiltration in this context includes both rainfall infiltration and transmission loss. Infiltration occurs only on the pervious area, including grids that do not receive rainfall while initial abstraction is applied to only the grid area that receives rainfall. If a grid element does not have rainfall, but receives runoff from upstream, infiltration is calculated without accounting for initial abstraction. For this test case, rainfall is applied to every grid element and there is no depth-area reduction specified. If IRAINBUILDING is set to 1, rainfall from areas blocked using *ARF* values are allowed to drain to adjacent grids, but runoff from upstream grids cannot enter grid elements that are totally blocked (*ARF* = 1). This is the case for this test model.

If IRAINBUILDING is set to zero, no runoff from rainfall is allowed from totally blocked grid elements. Rainfall from partially blocked grid elements (*ARF* < 1.0) is added to the rest of the grid element for runoff. Rainfall volumes from totally blocked grid elements are not included in the statistics reported in the SUMMARY.OUT file.

To verify WLII, the following input and output files are needed:

1. INFIL.DAT
2. FPINFILTRATION.OUT
3. ARF.DAT
4. CONT.DAT

Figure 5.2 SUMMARY.OUT: Surface Outflow Section

```
                    *** SURFACE OUTFLOW (ACRE-FT) ***

OVERLAND INFILTRATED AND INTERCEPTED WATER                     1.58 INCHES


          OVERLAND FLOW                                            WATER

WATER LOST TO INFILTRATION & INTERCEPTION                         651.02

FLOODPLAIN STORAGE                                               1829.56

FLOODPLAIN OUTFLOW HYDROGRAPH                                     883.42
                                                               ---------

FLOODPLAIN OUTFLOW, INFILTRATION & STORAGE                       3364.00

TOL FLOODPLAIN STORAGE                                             31.12

TOTAL SURFACE OUTFLOW AND STORAGE                                3364.00
```

The verification process is as follows:

1. Pervious area (PA(I)) for each grid element where I represents the grid number. Copy the *RTIMP* values for every grid from the INFIL.DAT file and paste into a spreadsheet.
2. PA(I) = (1-*RTIMP*(I))*GA
3. Using the INFIL.DAT file for *RTIMP* may not always provide the actual *RTIMP* value used. Keep mind that *FLO-2D* will do the following checks internally:
   A. If a partial *ARF* is assigned using XARF in CONT.DAT, then *RTIMP* is compared with it. If XARF > *RTIMP* then *RTIMP* is set equal to XARF.
   B. If a totally blocked grid element is assigned in *ARF*.DAT, *RTIMP* will be set to 1 if it is less than 1 in INFIL.DAT.

   C. If partial *ARF* factors are assigned using *ARF*(I) in *ARF*.DAT and that value is greater than *RTIMP*, *RTIMP* will be set to the *ARF*(I) value by the *FLO-2D* model at runtime. *ARF*'s are normally used to simulate buildings, so this data assignment forces the grid element surface area occupied by buildings to be impervious.
   D. Totaling the PA(I) for this example yields a PA of 4,844.83 acres.
4. Copy the total infiltration value for each grid from the FPINFILTRATION.OUT file, which is in units of feet or meters, into a column in the spreadsheet created under step 1 above. In a new column, multiply the PA(I) times the grid infiltration (I) and divide by 43,560 sq-ft/ac to obtain infiltration volume in ac-ft. Then sum that column to obtain total infiltration volume (TIV). For this example, the resultant volume is 599.44 ac-ft.
5. Copy the initial abstraction values (ABSTRINF(I), which are in units of inches, from the INFIL.DAT file into the spreadsheet. In a new column, divide the ABSTRINF(I) value by 12 to convert to feet then multiply by the total grid area to obtain the total volume of initial abstraction for each grid element. Then sum that column. For this example, the resultant volume is 51.65 ac-ft.
6. Add the total infiltration and initial abstraction volumes to obtain WLII:

   WLII = 599.44 + 51.65 = 651.09 ac-ft
7. This is a very close match to the reported volume of 651.02 ac-ft. The small difference is due to round-off differences between what *FLO-2D* uses and the spreadsheet values. *FLO-2D* is using double precision (16 significant digits) for all total volume numbers.
8. The spatial variability of *RTIMP* and *IA* from INFIL.DAT may preclude a simpler approach based on averaging the values. To test this, average *RTIMP* and *IA* from the INFIL.DAT file and the infiltration values from FPINFILTRATION.OUT, then perform a similar calculation. For this case, the average values are:

   $RTIMP_{avg} = 0.4778$

   $IA_{avg} = 0.0668$ inches

   $\text{Infiltration}_{avg} = 0.1135$ feet

   Total grid area = RA = 9,277.8696 acres

   WLII = (1-0.4778)*9,277.8696*0.1135 + 9,277.8696*0.0668/12 = 549.90 + 51.65
   = 601.54 ac-ft.

   This approach underestimates the total infiltration loss in this case. Some grid elements with the total surface area available for infiltration may infiltrate a lot of water while some partial surface area elements may not infiltrate very much water at all, which can result in this difference. The check should be done as described in steps 1-4 above.

OVERLAND INFILTRATED AND INTERCEPTED WATER (OIIW)

This value (1.58 inches) is the average volume of infiltrated water lost per grid element represented as a vertical column and includes the initial abstraction. It does not include the depression storage (*TOL*) value. The OIIW number represents the summation of the volume of loss on each grid element divided by the total area on which infiltration can occur. The spatially variable abstraction area may differ from the area of infiltration on an individual grid element because abstraction is also computed for building areas, infiltration is not. Therefore, the average infiltration is based on the grid element pervious area, and the average *IA* is based on the total grid element area. OIIW is verified as follows:

PA = 4,844.83306 acres

RA = 9,277.86961 acres

Total Infiltration Volume (TIV) = 599.44 ac-ft

Total Initial Abstraction Volume (TIAV) = 51.65 ac-ft

OIIW = ((TIV/PA) + (TIAV/RA))*12= (599.44/4844.83306 + 51.65/9277.86961)*12 = 1.55 inches.

This is a close check with the reported value of 1.58 inches. The small difference is due to round-off between *FLO-2D* and the spreadsheet values. *FLO-2D* is using double precision (16 significant digits) for all total volume numbers.

FLOODPLAIN STORAGE

This value (1,829.56 ac-ft) is the total volume of water remaining on the model surface at the end of the model simulation (TFPS). It represents the total flood depth remaining on each grid element and may be more or less than the tolerance value, *TOL*. If the volume of water entering a given grid element divided by the surface area is less than *TOL*, then the depth will not reach the *TOL* value. In ponded areas, the final flow depth can be greater than *TOL*. This volume can be verified using the depth data in feet in the FINALDEP.OUT file. The summation of the depth on each grid element, where the flow depth exceeded *TOL*, multiplied by the available grid element surface area represents the total volume left on the grid system. The available grid element surface area considered is as follows:

> The available grid element surface area is the total grid area minus the *ARF* area, and minus the area of street or *1D* channel. Outflow grid area is also not included. If *ARF* = 1, then there is no available surface area for that grid element.

This was done for this example and the result was 1,827.95 ac-ft, which is approximately 1% less than the reported value of 1,829.56 ac-ft. This is a reasonable check due to the use of reported depths rounded to 4 decimal places.

***TOL* FLOODPLAIN STORAGE**

This value (31.12 ac-ft) is the total volume of flow depth left on the available surface storage area (not the total area) at the specified *TOL* depth. Multiplying the *TOL* value for this model (0.004 feet) times the available storage area on each grid element, as defined above for FLOODPLAIN STORAGE, and then summing the product for every individual grid element and then dividing the sum by 43,560 sq-ft/ac yields 30.91 ac-ft. This result is approximately 0.7% less than the reported value of 31.12 ac-ft. This is a reasonable check. The final flow depth may be more or less than *TOL* as mentioned under FLOODPLAIN STORAGE above.

FLOODPLAIN OUTFLOW HYDROGRAPH

This value (883.42 ac-ft) is the total volume of outflow from the model through the outflow grid elements (TGOF). Since the inflow, infiltration and storage values are known and verified, the outflow volume is the inflow minus infiltration and storage. Storage for this check includes storage within the *SWMM* storm drain collection system at the end of the model simulation (TSDS). Using reported values:

TGOF=TIV – WLII – TFPS - TSDS

TGOF = 3,424.4 - 651.05 - 1829.86 - 42.48 = 901.01 ac-ft. This result is approximately 2% greater than the reported value. *SWMM* is reporting a 16.8% continuity error for this model, so this is a reasonable check. The outflow volume can also be computed from the reported hydrographs in the OUTNQ.OUT file. Using this approach yields a total outflow volume of 883.82 ac-ft, which is a reasonable check of the value reported in the SUMMARY.OUT file.

FLOODPLAIN OUTFLOW INFILTRATION AND STORAGE

This value (3,364.02 ac-ft) is the total of the reported WLII, TFPS and TSDS: 651.05 + 1,829.86 + 883.12 = 3,364.03 ac-ft. The value reported is correct.

TOTAL SURFACE OUTFLOW AND STORAGE

This value (3,364.00 ac-ft) is a repeat of FLOODPLAIN OUTFLOW INFILTRATION AND STORAGE.

5.2.3.3 FLO-2D Storm Drain Exchange Volume, ac-ft

The *SWMM* storm drain exchange volume summary statistics from the SUMMARY.OUT file are shown on Figure 5.3. Following Figure 5.3 are descriptions of each statistic listed. An extensive amount of time has been spent verifying these statistics. *FCDMC* is satisfied that the values presented are correct.

Figure 5.3 SUMMARY.OUT: Storm Drain Exchange Volume Section

```
          *** FLO-2D STORM DRAIN EXCHANGE VOLUME (ACRE-FT) ***

    Total Inflow To System

TOTAL INFLOW*                                                3267.158

*includes rainfall, inflow hydrographs and SD external inflow


    Storm Drain Inflow

SURFACE TO STORM DRAIN SYSTEM THROUGH INLETS                  278.061

SURFACE TO STORM DRAIN THROUGH OUTFALLS                        65.945
                                                            ---------

TOTAL (compare w/SWMM.rpt Wet Weather Inflow)                 344.006

    Storm Drain Outflow from Outfalls

STORM DRAIN TO SURFACE THROUGH OUTFALLS                       276.900

STORM DRAIN OUTFALL (OFF SYSTEM)                                0.000
                                                            ---------

TOTAL (compare w/SWMM.rpt External Outflow)                   276.900

    Storm Drain Return Flow to Surface

STORM DRAIN RETURN FLOW TO SURFACE THROUGH INLETS               6.809

STORM DRAIN STORAGE (PONDED FLOW INLETS)                        0.000

    Extracted from Storm Drain File (swmm.rpt)

WET WEATHER INFLOW                                            278.986

EXTERNAL INFLOW                                               119.662

EXTERNAL OUTFLOW                                              281.945

Total Storm Drain Storage (nodes+links)                        42.870

Continuity Error (%)                                           16.813
```

TOTAL INFLOW

This number includes all the inflow that enters the combined system including: rainfall, inflow hydrographs, and external inflow to the SD. It should be 3,424.40+119.66 = 3,544.06 ac-ft. The reported value is 3,267.16 ac-ft, which is incorrect. This bug has been reported to *FLO-2D* Software, Inc. and has been fixed.

SURFACE TO STORM DRAIN SYSTEM THROUGH INLETS

This is the total volume in ac-ft that enters the *SWMM* system through inlets (278.061 ac-ft).

SURFACE TO STORM DRAIN THROUGH OUTFALLS

This is the total volume in ac-ft that enters the *SWMM* system by backing up into storm drain outfalls from surface flows (65.945 ac-ft).

TOTAL

This is the total inflow to the storm drain system in ac-ft. It is the sum of SURFACE TO STORM DRAIN SYSTEM THROUGH INLETS and SURFACE TO STORM DRAIN THROUGH OUTFALLS (344.006 ac-ft). Check against the sum of the Wet Weather Inflow plus External Inflow minus Final Stored Volume minus Internal Outflow from the *SWMM*.RPT file (278.976 + 119.658 – 42.869 – 6.808 = 348.957 ac-ft). The 4.951 ac-ft difference is due to continuity error within *SWMM.*

STORM DRAIN TO SURFACE THROUGH OUTFALLS

This is the total volume in ac-ft leaving the storm drain system through outfalls to the floodplain surface (276.900 ac-ft). Check against External Outflow from the *SWMM*.RPT file (281.935 ac-ft) minus STORM DRAIN OUTFALL (OFF SYSTEM) (281.935 - 0.000 = 281.935 ac-ft). The small difference (5.035 ac-ft) is due to continuity error within *SWMM.*

STORM DRAIN OUTFALL (OFF SYSTEM)

This is the total volume in ac-ft that leaves the storm drain system through outfalls that are not returned to the floodplain grid surface (0.000 ac-ft).

TOTAL (compare w/SWMM.rpt External Outflow)

This is the total volume in ac-ft that leaves the storm drain system through outfalls including outfalls that discharge off the grid system (276.900 + 0.000 = 276.900 ac-ft). Check against External Outflow from the *SWMM*.RPT file (281.935 ac-ft). The small difference (5.035 ac-ft) is due to continuity error within *SWMM*.

STORM DRAIN RETURN FLOW TO SURFACE THROUGH INLETS

This is the total volume in ac-ft that leaves the storm drain system through inlets when the storm drain has a pressure head that exceeds the surface *WSEL* at the grid containing the inlet (6.809 ac-ft). Compare with Internal Outflow from the *SWMM*.RPT file (6.808 ac-ft).

STORM DRAIN STORAGE (PONDED FLOW INLETS)

This is the volume that is accumulated in the nodes when the pressure head exceeds the rim elevation but is below the *FLO-2D WSEL*, but because of the head comparison with the *FLO-2D WSEL*, the volume cannot return back to the surface. Recent changes to the *FLO-2D Pro* model with Build 15.10.13 have made this reporting item unnecessary and it will probably be removed in a future build.

WET WEATHER INFLOW

This value is the total inflow to the storm drain system in ac-ft from inlets and storm drain external inflow (278.986 ac-ft). This value is taken from the *SWMM*.RPT file, Wet Weather Inflow. This value should be close to the SURFACE TO STORM DRAIN SYSTEM THROUGH INLETS *FLO-2D* reported value (278.061 ac-ft). The small difference is due to continuity error within SWMM.

EXTERNAL INFLOW

This value is the total inflow to the storm drain system in ac-ft through outfalls (119.662 ac-ft). This value is taken from the *SWMM*.RPT file (119.658 ac-ft).

Total Storm Drain Storage (nodes+links)

This value is the total volume of water in ac-ft remaining within the storm drain system at the end of the simulation (42.870 ac-ft). This value is taken from the *SWMM*.RPT file, Final Stored Volume (42.869 ac-ft).

Continuity Error (%)

This is the *SWMM* volume conservation error reported in the *SWMM*.RPT file (16.813%). This value is taken from the *SWMM*.RPT file. It is calculated by dividing the difference between total inflow and total outflow plus storage by total inflow. For this example:

Total Inflow = 298.976 + 119.658 = 398.634 ac-ft

Total Outflow and Storage = 281.935 + 6.808 + 42.869 = 331.612 ac-ft

Difference = 67.022 ac-ft

Continuity Error = (67.022/398.634) * 100 = 16.813%.

5.2.3.4 Totals

The *SWMM* inflow and outflow volume summary statistics from the SUMMARY.OUT file are shown on Figure 5.4. The following are descriptions of each statistic listed.

TOTAL OUTFLOW FROM GRID SYSTEM

This statistic is the total outflow volume in ac-ft exiting the *2D* surface grid system through outflow grid elements (883.42 ac-ft).

TOTAL VOLUME OF OUTFLOW AND STORAGE

This statistic is the total volume in ac-ft of outflow from the grid system including grid and *1D* channel, outflow, infiltration and interception, SWMM outfall volume that discharges outside the grid, plus storage remaining on the grid system at the end of the simulation (3,369.65 ac-ft). For this example, it should equal the TOTAL SURFACE OUTFLOW AND STORAGE volume from the *** SURFACE OUTFLOW (ACRE-FT) *** section described above (3,364.00 ac-ft). The 5.65 ac-ft difference is due to continuity error within SWMM.

MAXIMUM INUNDATED AREA

This value is the total surface area of inundation in acres regardless of the time of occurrence for flow depths that are greater than the *TOL* parameter (7,735.05 acres). It is calculated by totaling the area of all grids that have a maximum flow depth greater than *TOL*, excluding all *ARF* area, including global *ARF* (XARF) assigned in CONT.DAT. This was verified in a spreadsheet resulting in a manually-computed maximum inundation area of 7,534.73 acres. The 200-acre difference was determined to be due to higher precision used by *FLO-2D* during the simulation than reported in the various maximum flow depth files, especially for values that only slightly exceed the TOL value(s). As a result, the model reporting in SUMMARY.OUT is considered to be more accurate than manual checks of the area of inundation.

THE MAXIMUM INUNDATED AREA (DEPTH > 0.5 FT)

This value is the total surface area of inundation in acres regardless of the time of occurrence for flow depths that are greater than 0.5 feet (1,199.94 acres). It is calculated by totaling the area of all grids that have a maximum flow depth greater than 0.5 feet, excluding any *ARF* area, outflow grid area, street and *1D* channel area. This was done in a spreadsheet resulting in a maximum inundation area of 1,200.22 acres. The difference of about 0.15%, which is due to round off error, is acceptable.

Figure 5.4 SUMMARY.OUT: Totals Section

```
                              *** TOTALS ***


TOTAL OUTFLOW FROM GRID SYSTEM                                          883.42


TOTAL VOLUME OF OUTFLOW AND STORAGE                                    3369.65


  SURFACE AREA OF INUNDATION REGARDLESS OF THE TIME OF OCCURRENCE:
  (FOR FLOW DEPTHS GREATER THAN THE "TOL" VALUE TYPICALLY 0.1 FT OR 0.03 M)

THE MAXIMUM INUNDATED AREA IS:                                     7735.05 ACRES

THE MAXIMUM INUNDATED AREA (DEPTH > 0.5 FT) IS:                    1199.94 ACRES
```

PERCENT LOSS (PL)

One of the rainfall-runoff and loss indicators that reviewers can apply is to estimate the overall percent loss due to infiltration and abstraction and then make a qualitative assessment on the reasonableness of the value. The first step is to make sure that the model total simulation time was long enough to move the majority of inflow and rainfall off the surface. To make this assessment, first look at the FLOODPLAIN STORAGE value. If it is small in relation to the total inflow volume that is a good indication that step 1 is met. If there is a large storage value, but there is also a large amount of known storage available on the surface including retention structures, levees, etc, then check a downstream main channel hydrograph for a small discharge rate at the tail. This can be done using predefined hydrographs written to the HYCROSS.OUT file. The model simulation time may need to be extended if indicated by these checks.

After the reviewer is satisfied that most of the inflow has moved through the system, calculate the percent loss (PL) as follows:

PL = WLII/TIV*100 = 651.05 ac-ft/3,424.40 ac-ft * 100= 19.01%

Keep in mind that this percentage also represents transmission losses in addition to rainfall infiltration and initial abstraction. For a highly urbanized watershed, this is a reasonable percentage, based on engineering judgment.

5.3 Hydrographs

Hydrographs are used to summarize output from the floodplain grids, outflow grids, *1D* channels, hydraulic structures, storm drain components, and levees and dams overtopping or breaching.

5.3.1 Floodplain Grids

Runoff hydrographs for specific locations are generated in *FLO-2D* for predefined locations using the FPXSEC.DAT input data file. A cross section is defined by a single line in that file. The *FLO-2D* grids to be summed are assigned along with the number of grids for the cross section and the single flow direction that will be used for all the grids. *FLO-2D* will determine the discharge leaving each grid for every reporting time step by summing the discharge for the preselected side plus the discharges for the two adjoining sides. Then the resultant discharges for the cross section grids are summed for each reporting time step.

The proper selection of grids for each cross section is very important. The cross section grids should be selected so they form a line perpendicular to the dominant flow direction. The grid octagon side corresponding to the dominant flow direction is assigned as the side to use for totaling the discharge for all grids in the cross section.

The resultant hydrograph for each cross section is written to the HYCROSS.OUT file. The identifier for each cross section hydrograph is the order number that it appears in the FPXSEC.DAT file. This file includes information regarding the top width, depth, velocity and discharge for each output time interval. This information can be viewed using the HYDROG program or graphed in MS Excel. The CROSSQ.OUT file contains the grid element hydrographs for each of the floodplain elements in the cross section.

5.3.2 Outflow Grids

The peak discharge leaving the model at each outflow element is written to the OUTNQ.OUT *FLO-2D* output file. Outflow grid peak discharges can also be written to a INFLOWx.DAT file to generate hydrographs that can be used as inflow hydrographs to a separate downstream *FLO-2D* model with a different grid system. This option is set in the OUTFLOW.DAT file. This information can be plotted and viewed using MS Excel.

5.3.3 Channels

The channel hydraulics output file, HYCHAN.OUT, contains a hydrograph for each channel element and includes time, water surface elevation, thalweg depth, average velocity, discharge, *Froude number*, wetted perimeter, hydraulic radius, top width, width-to-depth ratio, energy slope, bed shear stress and surface area. This information can be viewed using the *FLO-2D* HYDROG program or graphed in MS Excel. *FCDMC* has also developed GIS- and web-based tools that can be used to view these hydrographs and export the data to MX Excel.

5.3.4 Hydraulic Structures

The hydraulic structures output file, HYDROSTRUCT.OUT, contains a hydrograph for each structure and includes time and discharge. This information can be plotted and viewed using MS Excel. *FCDMC* has also developed *GIS*- and web-based tools that can be used to view these hydrographs and export the data to MS Excel.

5.3.5 Levees and Dams

If a levee or dam is overtopped during the simulation the discharge hydrograph overtopping the levee is written to the LEVEOVERTOP.OUT file. This file includes the time when the overtopping started, total discharge, discharge in each direction and peak discharge for each element of the levee. This information can be plotted and viewed using MS Excel.

If a levee or dam is breached using the functions in the LEVEE.DAT, the breach hydrograph is written to the LEVEE.OUT file. This file contains the time and direction of the failure, and for each levee element breached, the water surface elevation, failure width, and breach discharge for each output time interval. Also, if the structures are breached using the functions in the BREACH.DAT file, the breach hydrograph is written to the BREACH.OUT file. This file contains the time, direction, breach discharge, sediment discharge, sediment concentration, bottom width, top width and breach elevation. This information can be plotted and viewed using MS Excel.

5.3.6 SWMM

Several files are created to report the results of the storm drain system and these are:

- *SWMM*OUTFIN.OUT – reports the outfall discharge hydrograph to *FLO-2D*. This information can be plotted and viewed using MS Excel.
- *SWMM*QIN.OUT – reports the inflow discharge hydrograph to the storm drain system. This information can be plotted and viewed using MS Excel.
- *SWMM*.RPT – contains the discharge hydrograph for every drain inlet, outlet and conduit. The discharge hydrographs can be viewed using *GDS*.

- *SWMM*.OUT – this is a binary output file with results reported temporally and spatially that is read by the *EPA SWMM* GUI.

5.4 Flooding Depths, Elevations, and Profiles

There are several different files that provide results on depth, water surface elevation and profiles. These are summarized below: This information can be viewed using the programs included with *FLO-2D*, MAPPER PRO, MAPPER ++ and MAXPLOT. Additional information regarding these programs is included in Section 2.2 and the *FLO-2D* user manuals.

5.4.1 General

BASE.OUT – all-inclusive output file that contains flow depth, velocity, water surface elevation and discharge for either the channel or floodplain grid elements. It can be tailored to increase or decrease the amount of data written to it by changing the NOPRTFP and/or NOPRTC variables in the CONT.DAT input data file.

DEPTH.OUT – contains the maximum combined channel or floodplain flow depths for each grid element.

DEPTH*TOL*.OUT – contains the maximum combined channel and floodplain flow depths greater than the *TOL* value. Values less than the *TOL* value are set to zero.

5.4.2 Floodplain grids

DEPFP.OUT – contains the maximum floodplain flow depths.

FINALDEP.OUT – contains the final floodplain flow depths.

INTERGWS.OUT – contains the maximum floodplain water surface elevations for grid elements with flow depth greater than zero.

MAXWSELEV.OUT - contains the maximum floodplain water surface elevations for all grid elements.

5.4.3 Channels

DEPCH.OUT – contains the maximum channel flow depths.

CHANMAX.OUT – includes the maximum discharge and stage for each channel element and time of occurrence.

CHANWS.OUT – includes the maximum channel water surface elevation.

5.4.4 Profiles

Water surface profiles of the channel can be viewed using the program included with *FLO-2D*, PROFILES. The storm drain system profiles can be viewed by using the *SWMM* GUI.

Plot profile command in MAPPER ++.

5.5 Flood Hazard Identification

The MAPPER PRO program can be used to develop flood hazard maps. Instructions on how to develop the hazard maps are included in the white paper published by *FLO-2D* called *Hazard Maps.PDF* (FLO-2D Software, Inc., 2008).

5.6 Flow Depth Animations

Flow depth animations can be developed using the programs MAXPLOT, MAPPER PRO and *SWMM* GUI. Animations of the overland flow are created in MAXPLOT and MAPPER by setting the ITIMTEP variable in the CONT.DAT file. ITIMTEP is the time interval of the animation. The *SWMM* GUI shows an animation of the storm drain system and the controls are found on the Map Browser panel. The time interval is set using the reporting time step (REPORT_STEP in the *SWMM*.INP file). Animations rendered in Google Earth can be generated from the TIMDEP.OUT file using a computer program developed by *FCDMC*, which is available upon request to the *FCDMC* Engineering Division, Special Projects Branch.

5.7 *GIS* Integration

ESRI shapefiles can be imported into *FLO-2D GDS*, Mapper Pro and Mapper ++ to help create and review models. Shapefiles are also automatically created by Mapper Pro and Mapper ++ for all map plots and can be imported into *ArcGIS*. These files include:

- Maximum depth and final floodplain depth as a grid, contours and shaded contours
- Maximum velocity and final velocity as a grid, contours, shaded contours and vectors.

The *FCDMC* is developing a *GIS*- and web-based dissemination tool for *FLO-2D* projects. The planned capabilities include:

- Base layers such as aerial photographs and streets
- Input data such as grid ground elevation, roughness, and rainfall loss parameters
- Output data such as flow depth, velocity and discharge

- Temporal animations
- Hydrographs

The tool will be made available for internal and public use.

5.8 Summary and Conclusion

The *FLO-2D* Pro model has extensive output capabilities. The above is only a brief overview of the more important output files generated by the program. There is sufficient output generated in appropriate formats to meet the requirements of the *FCDMC*.

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6 APPROVAL AND STIPULATIONS

6.1 General

The detailed application testing documented in this report supports granting approval of the *FLO-2D* model for use on *FCDMC* projects for various purposes, including regulation, watershed planning, flood hazard mitigation, emergency action plans, and even for design where appropriate. This document is intended to ensure that *FCDMC* minimum requirements for hydrologic and hydraulic modeling are met. *FLO-2D* inherently requires a higher level of detail than traditional *1D* models. If the appropriate level of detail for an intended purpose is not available, then *FLO-2D* may not be an appropriate choice.

These tests verify that the various computations performed by *FLO-2D* components, including rainfall loss, hydraulic interaction between grid elements, *1D* channel hydraulics, hydraulic structures, obstructions to flow, the interface with the *EPA-SWMM* model and outputting of results, all individually function appropriately compared with current industry standards. The various components, when functioning as a whole, can reasonably reproduce actual storm results when applied using proper input data, level of detail, and engineering judgment. As with any hydrologic or hydraulic model whose components are technically correct, the results are only as good as the data used and the experience and judgment of the modeler.

Several older versions of the program were used in running the test models for the purpose of formalizing approval of *FLO-2D* for previous studies based on the older versions. The results of some of the multiple version testing are not documented in detail in this report to keep the report to a reasonable length. The results are similar to the model builds documented herein. The specific version approvals and stipulations for the use of each are listed in Section 6.2.

6.2 *FLO-2D* Version Approvals

6.2.1 *FLO-2D* 2009.06

All builds of *FLO-2D* 2009.06 are approved for hydrologic and hydraulic modeling. Approval stipulations are:

1. Only the *G&A* rainfall loss method is allowed. Extreme care should be taken when applying this method because *FLO-2D* computes both rainfall loss and transmission loss. The transmission loss component can easily be overestimated. Refer to Section 7.2.
2. Hydraulic structures are to only be modeled using the rating table or rating curve methods.

3. The *1D* channel component may be applied but care should be exercised in properly setting up the transitions between floodplain grids and the beginning and end of each *1D* channel segment. Also, very wide cross sections may result in isolated grid elements within the channel not being excluded from the model.

6.2.2 *FLO-2D* Pro Build 12.10.02 through 15.07.12

1. Only the *G&A* rainfall loss method is allowed. Extreme care should be taken when applying this method because *FLO-2D* computes both rainfall loss and transmission loss. The transmission loss component can easily be overestimated. Refer to Section 7.2.
2. Hydraulic structures are to only be modeled using the rating table or rating curve methods.
3. The *1D* channel component may be applied but care should be exercised in properly setting up the transitions between floodplain grids and the beginning and end of each *1D* channel segment. Also, very wide cross sections may result in isolated grid elements within the channel not being excluded from the model.
4. The *SWMM* storm drain component was under development for these builds. Models using the storm drain component should be based on Build 15.10.13 for final results.

6.2.3 *FLO-2D* Pro Build 15.10.13 and Newer

1. Only the *G&A* rainfall loss method is allowed. Extreme care should be taken when applying this method because *FLO-2D* computes both rainfall loss and transmission loss. The transmission loss component can easily be overestimated. Refer to Section 7.2.
2. Hydraulic structures may be modeled using the rating table, rating curve, or general equations methods.
3. The *1D* channel component may be applied but care should be exercised in properly setting up the transitions between floodplain grids and the beginning and end of each *1D* channel reach.
4. Application of the *SWMM* storm drain component is allowed.

6.2.4 Future Program Releases

Program releases after build 15.10.13 must receive prior approval by the *FCDMC* Engineering Division before application on *FCDMC* projects or for modeling that will require *FCDMC* approval. *FCDMC* will run the series of test models discussed herein, and review the results and compare with known benchmark data before approving a new build for application.

7 APPLICATION RECOMMENDATIONS

7.1 Introduction

This section includes recommended approaches and procedures for applying various *FLO-2D* components. These are based on experience gained by application of the *FLO-2D* model on *FCDMC* projects over the past several years, and from development of the various test models documented herein. It is not intended to be a complete application guide. The intent is to provide lessons learned. The user is encouraged to explore new and cost-effective methods, tools and procedures for developing the large and complex models that *FLO-2D* is capable of. Passing new ideas and methods along to the *FLO-2D* community is encouraged.

7.2 Procedures for Handling Rainfall Losses

7.2.1 General

The volume of water routed on the surface is perhaps the most important aspect of the *2D* surface model. *FCDMC* uses the *G&A* method for estimating rainfall and transmission losses, which controls how much volume is on the surface. Similar to *HEC-1* and *HEC-HMS*, *FLO-2D* is currently only capable of simulating the surface soil as a single horizon. Multiple soil horizons are not accounted for. Originally, the *FLO-2D* model treated the surface soil horizon (versions up through 2009.06) as an unlimited reservoir. The model would continue to infiltrate water as long as there was flow depth on the surface in excess of *TOL* after *IA* was met. This could significantly over estimate losses because of the spatial variability of physical soil properties and the case where impermeable soil horizons exist. With the addition of a spatially-variable *LID* option, the user has significantly more control over the volume of water infiltrated.

Other parameters and settings affecting rainfall and transmission losses discussed in this section are *IA*, *TOL*, *XKSAT*, *DTHETA*, Shallow *n*, and *RTIMP*.

7.2.2 Limiting Infiltration Depth

The *LID* option allows for setting a soil horizon depth limit. When infiltration penetrates to the assigned *LID* for a grid element, and *DTHETA* is filled, infiltration is no longer allowed to occur for that grid element. To apply this option, the following should be considered:

1. The *NRCS SSURGO* soil databases that the *FCDMC* uses to estimate *G&A* parameters should be consulted to determine if there is an impervious soil horizon shallow enough to warrant being

included in the model. Then, if the impermeable horizon is for a component soil that is only a portion of a Soil Map Unit (*SMU*), engineering judgment should be used to determine the spatial extent. This may warrant a field investigation. Otherwise, the depth of the impermeable soil horizon should be assigned for the entire extent of the *SMU*, also based on engineering judgment. An Excel spreadsheet can be obtained upon request from the *FCDMC*, Engineering Division, Special Projects Branch that includes a listing of soil horizons for every *SMU* within Maricopa County.

2. *LID* can be applied as a calibration tool. Refer to Section 7.7.

7.2.3 *IA* and *TOL*

IA is the initial abstraction parameter and *TOL* is the *FLO-2D* control setting that is the minimum flow depth on a grid element before volume can be exchanged with adjacent grid elements. The two are similar and complimentary. For rainfall-runoff models, *TOL* is recommended to be set to a small value. *FCDMC* recommends a minimum setting of 0.004 feet or about 0.05 inches. Since *TOL* effectively functions as a form of *IA*, the *FCDMC* estimate of *IA* for each grid element should be adjusted by subtracting the assigned *TOL* value.

For *FCDMC* projects, *IA* is normally assigned using a Surface Feature Characterization. Refer to Section 7.2.7 for a discussion. *IA* can also be used as a calibration parameter. Refer to Section 7.7.

7.2.4 XKSAT

The *FCDMC* standard rainfall loss method includes an adjustment to bare ground *XKSAT* to account for the effects of soil crusting, ground cover and vegetative canopy cover. Refer to the *DDM Hydrology* (FCDMC, 2013a). This adjustment is not recommended for use with the *FLO-2D* model. Instead, bare ground *XKSAT* should be used. The *FLO-2D G&A* implementation computes transmission losses in combination with rainfall loss. The vegetative correction may not be appropriate for long term infiltration other than rainfall loss; therefore, until more is known about these processes, *FCDMC* has elected to not account for *XKSAT* adjustment due to vegetation effects for *2D* hydrologic modeling.

The *FCDMC* standard rainfall loss method also includes a recommended method for log-area-averaging *XKSAT* for *1D* hydrology models. The *FLO-2D GDS* program applies this approach when computing the bare ground *XKSAT* estimate for each grid element. When other methods such as *GIS* are used, application of the log-area-averaging procedure is not necessary. *XKSAT* may be assigned by overlaying the *NRCS* soil survey polygons over the grid and assigning the value of *XKSAT* from the *SMU* at the center of each grid element. Log-area-averaging is not necessary because the grid size is very small in comparison with typical *SMU* polygon areas. *XKSAT* is not recommended to be used as a model calibration parameter unless the other calibration options are not sufficient. If this becomes necessary, obtaining field measured *G&A* parameters may be required.

7.2.5 DTHETA

DTHETA should be applied as recommended in the *DDM Hydrology* (FCDMC, 2013a).

For 1D Channels: To apply the *FCDMC* definition of *DTHETA*, set the *FLO-2D* parameters *SATI*, *SATF* and *POROS* as follows: *SATF* = 1.0 and *SATI = SATF - DTHETA*, where *DTHETA* is the average value for all channel reaches or elements in the model, and *POROS* = 0 or 1. When *POROS* is set equal to zero (0), *FLO-2D* does not multiply *DTHETA* by POROS. POROS can also be set equal to one (1) when volumetric *DTHETA* is used. *FLO-2D* will multiply *DTHETA* by one (1), which has the same effect as using zero (1). Refer to Section 2.3.3.5.

For Floodplain Grids: To apply the *FCDMC* definition of *DTHETA*, set the *FLO-2D* parameters *SATI*, *SATF* and *POROS* as follows: *SATF* = 1.0 and *SATI = 0*, and *POROS* = 0 or 1.

DTHETA can be used as a calibration parameter, particularly when modeling actual storms and the soil moisture condition can be estimated. Refer to Section 7.7.

7.2.6 PSIF

PSIF should be applied as recommended in the *DDM Hydrology* (FCDMC, 2013a). *PSIF* should not be used as a calibration parameter except under the same conditions as *XKSAT*.

7.2.7 RTIMP

RTIMP is a critical input parameter. Watersheds with large areas of natural and/or developed impervious area must have as accurate a grid-based estimate of *RTIMP* as possible. For natural watersheds, the minimum approach is to use the *NRCS* Soil Survey *RTIMP* values provided by *FCDMC*. These estimates should be field verified for reasonableness. The modeler may need to estimate polygons of various levels of impervious area based on aerial photography in combination with field reconnaissance/survey information. *RTIMP* estimates for use with *FLO-2D* are considered 100% effective or directly connected hydraulically. Since the *FLO-2D* grid element is small in relation to the watershed as a whole, this is a valid assumption. In contrast, the *FCDMC 1D* hydrology models allow for reducing the amount impervious area to an estimate of the impervious area that is directly connected to the watershed outlet or concentration point.

FCDMC prefers that *RTIMP* for urban areas be estimated using a detailed Surface Feature Characterization. This consists of a detailed *GIS* polygon-based feature class of the surface type classes listed in Table 7.1. The level of detail will vary depending on the availability of supporting data. The polygons for the various type classes are typically generated using the following methods:

1. From cartography line work created by the aerial mapping company as a part of the contract to generate a detailed three dimensional (*3D*) surface of the study area.
2. From polygons of various features digitized manually from available ortho-rectified aerial photographs. This method is normally used to supplement item 1 where development has occurred since the aerial mapping flight date or to add features not generated by the mapping contractor.
3. From raster to vector software such as *Feature Analyst* (TEXTRON Systems, 2016) using available ortho-rectified aerial photographs.

This approach can provide an accurate estimate of the actual imperious area within each grid element. This information is also used for assigning initial values of *IA*, *DTHETA* and *n*-value for each grid element. The 'Priority' field is used for addressing the situation where type class polygons of one type overlay polygons of other type classes. The priority establishes the order for clipping the various type class polygons using *GIS* tools to create a complete surface feature characterization without any overlaps or gaps.

The other areas remaining unclassified in a study area are assigned type classes using *NRCS* Soil survey polygons for the geomorphic features listed in Table 7.2. These are generalized surface types that can be adjusted manually as necessary. *RTIMP* values for these class types are assigned from the *NRCS SMU* data.

Table 7.1 Surface Feature Characterization Type Classifications

CLASS ID	Type Class	Description	*IA*	*RTIMP*	InitSat	*n*	Type	Priority[1]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0	Natural High Vegetation	Trees	0.10	0	dry	0.065	Natural	16
1	Natural Medium Vegetation	Shrubs and brush	0.10	0	dry	0.030	Natural	18
2	Natural Low Vegetation	Grass and low shrubs	0.10	0	dry	0.055	Natural	20
3	Urban High Vegetation	Trees	0.10	0	normal	0.065	Urban	15

Table 7.1 Surface Feature Characterization Type Classifications

CLASS ID	Type Class	Description	*IA*	*RTIMP*	InitSat	*n*	Type	Priority[1]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
4	Urban Medium Vegetation	Shrubs and bushes	0.10	0	normal	0.055	Urban	17
5	Urban Low Vegetation	Lawns and low shrubs	0.10	0	normal	0.045	Urban	19
6	Mountain Bare Ground	Mountain bare ground	0.25	0	dry	0.050	Natural	22
7	Hillslope Bare Ground	Hillslope Bare Ground	0.15	0	dry	0.045	Natural	23
8	Desert Rangeland Bare Ground	Desert Rangeland Bare Ground	0.35	0	dry	0.040	Natural	24
9	Urban Bare Ground	Urban Bare Ground	0.20	0	dry	0.035	Urban	21
10	Desert Landscaping permeable	Desert landscaping without an impermeable membrane	0.20	0	normal	0.040	Urban	14
11	Desert Landscaping impermeable	Desert landscaping with impermeable membrane	0.10	95	saturated	0.040	Urban	8
12	Wash Bottom	Natural wash and river bottoms	0.10	0	dry	0.035	Natural	12
13	Concrete	Sidewalks, curb, patios	0.05	98	normal	0.016	Urban	4
14	Asphalt	Streets and parking lots	0.05	95	normal	0.020	Urban	5
15	Buildings	Physical structures that are flow obstructions	0.05	95	normal	0.024	Urban	1
16	Shade Structures	Parking covers, canopies	0.05	98	normal	0.035	Urban	6

Table 7.1 Surface Feature Characterization Type Classifications

CLASS ID	Type Class	Description	*IA*	*RTIMP*	InitSat	*n*	Type	Priority[1]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
17	Water	Lakes, canals, ponds	0.00	100	saturated	0.040	Urban	7
18	Swimming Pools	Pools	3.00	100	saturated	0.040	Urban	2
19	Rock 100	Large extents of solid rock outcrop	0.25	95	dry	0.060	Natural	10
20	Rock 85	Broken fractured rock outcrop	0.25	80	dry	0.050	Natural	11
21	Unpaved road	Gravel and dirt roadways and shoulders	0.10	50	dry	0.026	Urban	9
22	Agricultural	Farm fields	0.50	0	normal	0.060	Urban	13
23	Rock Riprap	Rock riprap lined channel banks and/or bed	0.25	95	dry	0.065	Natural	3

[1] Higher numbered priorities are erased with lower numbered priorities. Erase the largest number (23) with the next highest number (22). Merge the two, then erase with 21. Merge 21 with 22 & 23, etc.

Table 7.2 Geomorphic Feature Type Classifications

Geomorphic Feature	CLASS_ID	Type	Type_Class	*IA*	*RTIMP*	InitSat	n
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
alluvial fans	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\basin floors	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\channels\terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040

Table 7.2 Geomorphic Feature Type Classifications

Geomorphic Feature	**CLASS_ID**	**Type**	**Type_Class**	***IA***	***RTIMP***	**InitSat**	**n**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
alluvial fans\fan terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\flood plains	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\flood plains\stream terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\flood plains\terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\plains	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\plains\stream terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\ridges	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\stream terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
alluvial fans\terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
basin floors	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
basin floors\fan terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
dunes	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
fan terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
fan terraces\hills	7	Natural	Hillslope Bare Ground	0.15	0	dry	0.045
fan terraces\stream terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040

Table 7.2 Geomorphic Feature Type Classifications

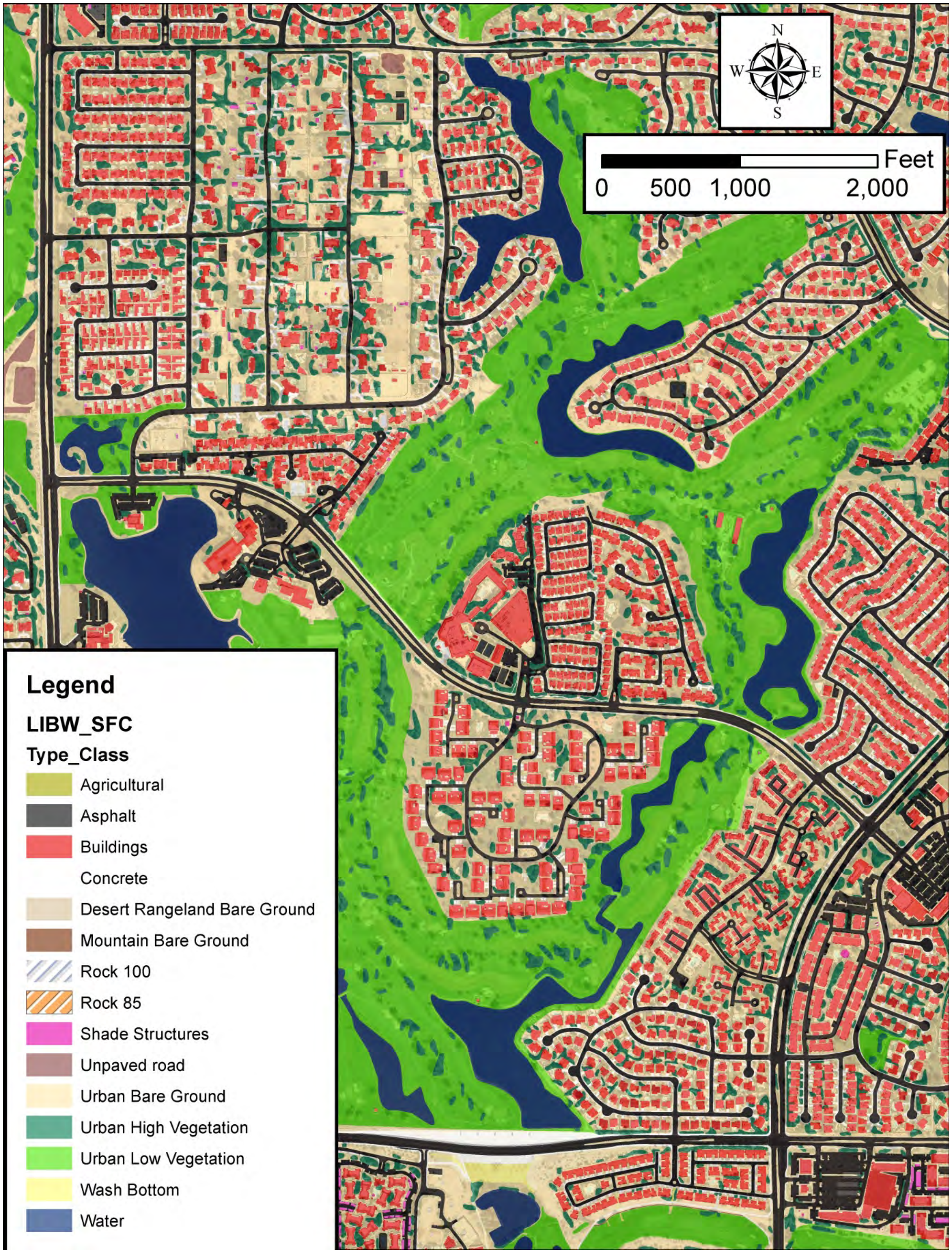
Geomorphic Feature	CLASS_ID	Type	Type_Class	*IA*	*RTIMP*	InitSat	n
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
fans\flood plains	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
flood plains	12	Natural	Wash Bottom	0.1	0	dry	0.035
flood plains\stream terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
flood plains\terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
flows\hills	7	Natural	Hillslope Bare Ground	0.15	0	dry	0.045
flows\hills\mountains	6	Natural	Mountain Bare Ground	0.25	0	dry	0.050
hills	7	Natural	Hillslope Bare Ground	0.15	0	dry	0.045
hills\lava flows	7	Natural	Hillslope Bare Ground	0.15	0	dry	0.045
hills\lava flows\mountains	6	Natural	Mountain Bare Ground	0.25	0	dry	0.050
hills\mountain slopes	6	Natural	Mountain Bare Ground	0.25	0	dry	0.050
hills\mountains	6	Natural	Mountain Bare Ground	0.25	0	dry	0.050
hills\pediments	7	Natural	Hillslope Bare Ground	0.15	0	dry	0.045
hillslopes	7	Natural	Hillslope Bare Ground	0.15	0	dry	0.045
hillslopes\mountain slopes	6	Natural	Mountain Bare Ground	0.25	0	dry	0.050
hillslopes\mountain slopes\pediments	6	Natural	Mountain Bare Ground	0.25	0	dry	0.050
hillslopes\pediments	7	Natural	Hillslope Bare Ground	0.15	0	dry	0.045

Table 7.2 Geomorphic Feature Type Classifications

Geomorphic Feature	CLASS_ID	Type	Type_Class	*IA*	*RTIMP*	InitSat	n
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
mountain slopes	6	Natural	Mountain Bare Ground	0.25	0	dry	0.050
mountains	6	Natural	Mountain Bare Ground	0.25	0	dry	0.050
pediments	7	Natural	Hillslope Bare Ground	0.15	0	dry	0.045
plains	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
plains\stream terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
plains\terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
stream terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040
terraces	8	Natural	Desert Rangeland Bare Ground	0.35	0	dry	0.040

An example excerpt from a Surface Feature Characterization from the Lower Indian Bend Wash *ADMP* is shown on Figure 7.1. The *FLO-2D* model for this *ADMP* uses a 20 foot grid size. The surface features are represented at a high degree of accuracy including building obstructions.

Figure 7.1 Example Surface Feature Characterization



7.3 Obstructions to Flow

Obstructions to flow for most *FCDMC FLO-2D* models consist of buildings and walls. General practices for the *FCDMC* approach to modeling these obstructions using *FLO-2D* are described in the following sections.

7.3.1 Buildings

In general, buildings should always be modeled as obstructions to flow unless there is strong reason not to. *FCDMC* recommends that buildings be modeled using the *ARF* input parameter. Buildings that are larger than the grid size may completely enclose entire grid elements. The completely enclosed grid elements should have an *ARF* assignment of 1 using the 'T' record in the *ARF*.DAT input data file. The adjacent grids along the edge of a building that are only partially covered by the building should have a partial *ARF* assignment in the 2nd field of the *WRF* record. The *WRF* values for these grids should be assigned a value of 0.0 in fields 3 through 10. If the model is a rainfall-runoff model, then the building rainfall switch (IRAINBUILDING) should be set to 1. Refer to the *FLO-2D* Pro Data Input Manual (FLO-2D Software, Inc., 2015d) for additional guidance. If runoff from a building or portions of a building need to be routed to a downspout location, guidance is provided in the supplemental *FLO-2D* handout document *Building Roof Runoff with Downspouts* (FLO-2D Software, Inc., 2015a). The elevations for the building grids should be carefully evaluated to ensure positive drainage for the roof to the desired side of the building and to remove any ponding.

For *FCDMC* projects, flow obstructions caused by buildings are normally assigned using a Surface Feature Characterization. Refer to Section 7.2.7 for a discussion.

7.3.2 Walls

FCDMC recommends that walls that are solid obstructions (masonry and concrete walls) be modeled. The approach should be to model the walls using the *FLO-2D* levee component. For recent *FCDMC* projects, wall alignments are provided as *GIS 3D* polylines. Under *FCDMC* specifications, these lines are generated by the mapping contractor as a part of the topographic mapping used for the project. The walls may be set to fail when the flow depth against the wall reaches a specified depth. Modeling of walls to fail may or may not be written in to the project scope depending on the goals of the study. Generally, *FCDMC* asks that not all walls be added into the *FLO-2D* input data files at one time. The normal procedure is to run the model first without walls and then use the flow pattern results to target specific walls that need to be included. This can be an iterative process because the diversions caused by added walls may result in the need to add additional walls. Simulating a large number of walls can slow the model significantly, which is why this approach is used.

7.4 Manning's *n*-Values

7.4.1 General

Appropriate definition of surface roughness is a critical part of *FLO-2D* model development. The following sections set forth *FCDMC* guidance for *n*-value selection and application.

7.4.2 Depth-Variable *n*-Value Approach

FCDMC recommends use of the depth-variable *n*-value option for most *FLO-2D* models. A possible reason for not using this option is discussed in Section 7.4.3.

7.4.3 Shallow *n*

FCDMC recommends that the Shallow *n* parameter be applied for watersheds that are predominately natural. Learn more about the Shallow *n* parameter in *Shallow Flow Roughness and TOL Values* (FLO-2D Software, Inc., 2015e). Care should be taken to not apply too high a value for rainfall-runoff models as Shallow *n* can significantly increase transmission loss by causing unrealistically low velocities in areas with small flow depths. In general, Shallow *n* can be assigned as follows, but the user should apply these using engineering judgment based on the physical characteristics present:

1. Highly developed urban areas (little natural surface remaining): 0.10 to 0.12.
2. Natural areas, desert rangeland: 0.12 to 0.15
3. Natural areas, hillslope: 0.12 to 0.18
4. Natural areas, mountain: 0.15 to 0.20

Shallow *n* can be used as a calibration parameter as follows: 1) reduce the value if the model results lag behind measured or the measured runoff volume is greater than the model results, and 2) increase the value if the measured results lag the model results or the model runoff volume is greater than the measured. Adjustments to Shallow *n* for model calibration should only be attempted after applying the *LID* and *IA* calibration options, and/or improving model routing times by appropriate application of *1D* channels or multiple channels.

The Shallow *n* option may be turned off for highly urbanized watersheds when model timing is lagging behind measured, the majority of the conveyance areas are paved, and:

- Appropriate application of other hydraulic measures to improve model response such as the use of *1D* channels and/or multiple channels does not resolve the timing issue,
- Adjustment of grids defining the street crowns and gutters to provide uniform longitudinal slopes does not resolve the timing issue,
- Use of a low Shallow *n* setting of 0.1 does not resolve the timing issue, and
- Flow depths are mostly shallow and the street conveyance system has unreasonably low velocities when compared with normal depth street hydraulic calculations.

Shallow *n* is turned off by turning off the depth-variable *n* option, which is done by setting the AMANN control parameter to '-99.'

7.4.4 Spatially-Varied *n*

Manning's *n* should be spatially-varied for all models. The standard *n*-value assignment, assuming the depth-variable *n*-value option is turned on, is for flow depths of 3 feet and greater. Typically, *FLO-2D n*-value assignments should be greater than are normally used for *1D* hydraulic modeling because of the unsteady and non-uniform flow contribution between elements and the flow not being oriented in one direction. *FCDMC* has established *n*-values for the various surface feature classes described in Section 7.2.7, as listed in Table 7.1 and Table 7.2. These are initial starting values that should be adjusted by the modeler based on actual surface conditions within the study area. As model development progresses and initial output results become available, the modeler should carefully examine *n*-value adjustments dynamically made by the *FLO-2D* program. These are listed in the ROUGH.OUT file primarily as a result of the limiting Froude number option application. The *FLO-2D* program will increase or decrease the *n*-value to reach the target Froude number. Refer to *FLO-2D Limiting Froude Number Application Guidelines* (FLO-2D Software, Inc., 2010) for more information.

The values in the ROUGH.OUT file can be used in conjunction with information in the TIME.OUT file to revise the assigned *n*-value for critical grid elements that are slowing down the model. It is not recommended to change all of the *n*-value assignments to match the maximum *n*-value listed in the ROUGH.OUT file. Instead, work on the grid elements slowing the model, including adjacent grids, and the grid elements that are being reset to unreasonably high values and their adjacent grid elements. Select and assign a new more reasonable value somewhere in between the original setting and the maximum setting and rerun the model. This is an iterative process. Keep in mind that sometimes the maximum *n*-value for a grid in the ROUGH.OUT file could be occurring early in the simulation on a steep cross slope perpendicular to a wash. The *n*-value adjustments are made for that slope/conveyance condition. Later in the simulation, when flow is dominated by runoff in the channel perpendicular to the cross slope, the adjusted *n*-value may be too high for the slope/conveyance relationship of the main channel. This is only one example that illustrates why changing all of the *n*-values in the TOPO.DAT or FPLAIN.DAT files to match the maximum *n*-values from the ROUGH.OUT file can lead to complications.

7.4.5 *n*-Value Adjustment for Deep Ponded Areas

Deep water with slow velocity in reservoirs, detention basins or other ponded features represents a unique condition for flood routing in the *FLO-2D* Pro Model. Refer to *FLO-2D Pro Reservoir Routing and Ponded Flow* (FLO-2D Software, Inc., 2014b) for a full description of the issues and technical guidance. In conjunction with application of the DEP*TOL* parameter in the TOLER.DAT input data file, it is recommended to increase the Manning's *n*-value for grids in deep ponded areas. The values in Table 7.3 provide a starting point for making such adjustments. Applying the DEPTOL parameter may help with stability issues associated with large ponded depth areas.

Table 7.3 Ponding Depth *n*-Values

Ponding Depth Range	Base *n*-value
(1)	(2)
5-8 ft	0.080
8-10 ft	0.100
10-15 ft	0.200
15ft and greater	0.300

7.5 Courant Number

The *FLO-2D* control variables COURANTFP, COURANTC, and COURANTST allow the user to specify the Courant Number stability parameter for floodplain grids, *1D* channels, and street elements, respectively. The Courant Number is used to limit the time step magnitude to avoid surging and still allow large enough time steps to complete the simulation in a reasonable timeframe. Guidance in the application of the Courant Number is provided in *User Assigned Courant Number in TOLER.DAT for Enhanced Model Numerical Stability* (FLO-2D Software, Inc., 2012b). *FLO-2D* models without a high degree of complexity can normally be run with a Courant Number setting of 0.6. However, models with complex hydraulic structures, *1D* channels, levees and/or storm drains may require a reduced Courant Number to avoid numerical surging. Refer to Section 4.2.5.19 for examples. The modeler should carefully examine maximum velocities, and water surface profiles and output hydrographs for inappropriate oscillations, that may indicate surging and the need for lower Courant number settings.

7.6 DEPTOL and WAVEMAX

The DEPTOL and WAVEMAX control parameters should normally be set to zero (0) for *FCDMC* models. Refer to the user manuals for guidance.

7.7 Calibration Techniques

7.7.1 General

FLO-2D model results should be checked for reasonableness, verified against documented flood depths from actual storm events, and calibrated whenever sufficient gage data is available. Guidance for each of these approaches is provided in the following sections.

7.7.2 Checks for Reasonableness

Checks for reasonableness include, but are not limited to, the following:

- Plotting and examination of hydrographs from floodplain cross section, *1D* channel cross sections, hydraulic structure hydrographs, storm drain inlet, pipe and outflow hydrographs.
- Comparison of peak discharge results with indirect methods. Refer to Chapter 8 of DDM Hydrology (FCDMC, 2013a). These indirect method checks should only be applied for mostly natural tributary watersheds similar to the gaged watersheds the data comes from and where the watershed area can be accurately estimated.
- Where appropriate, general hydrograph shape and proportion of volume (roughly 1/3 of volume before the peak and 2/3 after the peak).
- Summary volumes and statistics in the SUMMARY.OUT file.
- Water surface profiles at critical locations or points of concern within the study area.
- Examination of retention basins to ensure that flow intended to reach each basin actually does.
- Flow direction vectors in and around flow obstructions to ensure they are coded correctly.
- Significant areas of ponded water to ensure that the grid elevations correctly represent the actual storage.
- Grid elevations along street center lines and gutters to ensure that the street systems are properly represented.
- Grid elevations along embankments to ensure that they are properly represented.
- Maximum velocities.

7.7.3 Verification/Confidence Checks

FCDMC project *FLO-2D* models should be run using at least one actual storm *GARR* data set. The results should then be checked for reasonableness against actual depth data obtained from available sources, including but not limited to:

- Photographs of flooding during the storm documented in newspaper articles, field observations by *FCDMC* personnel, residents in the area, photos submitted through the *FCDMC* *Report A Flood* mobile tool app (http://gis.fcd.maricopa.gov/raf/) and other sources.
- Municipal maintenance records.
- Municipal storm flood complaints databases.
- Aerial photographs taken during or immediately after the flood.
- Stream gage and/or retention basin stage gage data.
- Visible high water marks in channels, culverts, culvert inlets, retention basins, and on bridge piers.

These comparisons should be documented in the technical report for the project and possibly used to help calibrate the model if the results show trends that can be addressed through model parameter adjustments.

7.7.4 Calibration Parameters

Output that should be targeted for calibration in the preferred order of priority includes:

1. Total runoff volume on the surface.
2. Flow depths at critical locations.
3. Runoff hydrograph timing, peak discharge and runoff volume.

The following are *FLO-2D* input parameters that can be used in model calibration, listed in the preferred order of priority for adjustment:

1. Grid elevations. Manual adjustment of grid elevations at critical locations in order to more accurately simulate the actual ground surface is a very powerful calibration tool. Improper elevation assignments can have a large effect on storage and therefore on distribution of volume on the surface. The first step in model calibration is refinement of grid elevations to make sure that water is directed appropriately, including ensuring that proper flow amounts reach storm drain inlets, hydraulic structures, basins and channels.
2. *LID*. Spatially-varied *LID* settings can be adjusted up or down globally if appropriate, or for specific features such as the base of natural channels or areas of *SMU*s where a controlling soil horizon is known to exist. This is a very powerful tool for accomplishing model calibration goals.
3. *IA*. Spatially-varied *IA* can be used to control the volume on the surface or at specific locations within the study area. Because there is a limited range within this parameter can be varied (0.05” to 1”), it has a limited effect on calibration. It can be used to correct small variances in total volume on the surface.
4. Manning’s *n* and Shallow *n*. Adjustment of surface roughness is a powerful calibration tool, particularly for affecting timing of runoff. In general, make adjustments to Manning’s *n* first to

address model stability issues and overall watershed timing, and Shallow *n* 2nd to address volume and further address timing issues. Hydrograph lag times can be significantly increased or decreased through judicious *n*-value adjustments. Also, both *n*-values and Shallow *n* can be used to affect the total volume on the surface because they affect the length of time flow depth in excess of *TOL* remains on grid elements. The longer the duration the more infiltration occurs.

5. *1D* Channels. The proper use of *1D* channels can dramatically improve the timing of runoff. Using floodplain grids to simulate channel flow can be problematic when the channel widths are less than the grid width. Adding in prismatic *1D* channels to simulate constructed or even fairly uniform natural channels for these cases can significantly improve watershed response and enable a better match with measured hydrographs. Their use can also help with modeling channels that are greater than the grid width, but the effects will not have as large an impact on model results.
6. Multiple channels. The multiple channel option can be used in a similar manner to the *1D* channel option described above. Refer to Section 7.11.5 for limitations on its use.
7. *DTHETA*. Adjustment of the *DTHETA* parameter can be useful for reproducing measured results for actual storms. When creating actual storm models, the initial moisture content of the soil is a very important input parameter. However, the selection of what soil moisture condition to use for a design storm model may need to be different than used for the storm reconstitution model.
8. *XKSAT*. Adjustment of the *XKSAT* parameter could be a powerful tool for adjustment of the total volume on the surface but should only be used as a last resort. The majority of calibration issues can be resolved using methods 1 through 7 above. *FCDMC* prefers to use its standard *XKSAT* values whenever possible unless field- measured *G&A* parameters are available.

7.7.5 Calibration: Gage Data Available

When stream flow or stage gage(s) exist within the model domain, the measured data should be used, as a minimum, to verify and provide confidence checks of the model results. The recommended procedure is to identify actual storms that impacted the *FLO-2D* model domain and produced valid measureable runoff hydrographs at the gages. The measured results should undergo a quality control check by staff responsible for the gage data to ensure the measured results are valid. Ideally, there should be more than one storm available. It is useful to have at least one high intensity summer thunderstorm and one longer duration winter storm. The spatially- and temporally-varied rainfall data should be obtained as described in Section 4.3 and used to create the RAINCELL.DAT *FLO-2D* input data file. That file is used to impose a moving storm on the model domain. The model should be run using the actual storm rainfall and the results compared with the measured data. This includes comparisons with any verification/confidence check data gathered as part of the data collection efforts described in Section 7.7.3.

If there is a reasonable comparison, then no calibration is necessary. If there are significant differences, then model calibration may be required. Assuming that the grid elevations have already been checked and adjusted as described in Section 7.7.4, general guidance for the approach to model calibration is as follows:

7.7.5.1 Case 1: Less volume than measured and hydrograph shape similar to measured

This case generally requires a more global approach to decreasing rainfall and transmission loss. If the volume difference is fairly small, evaluate if the *TOL* value and *IA* could be affecting the difference. If so, and *IA* can be reduced uniformly across the model domain and still be within an acceptable range, try that approach first. If the difference is too large for that approach, then try reducing the *LID* globally. If these two approaches move the IA and/or *LID* values into the unreasonable range, consider reducing Shallow *n*. Evaluate the impervious area. Changes to the impervious area may affect the hydrograph shape, so adjustments to *RTIMP* should be carefully evaluated. Finally, for the worst case, try turning off Shallow *n*.

7.7.5.2 Case 2: More volume than measured and hydrograph shape similar to measured

Follow the same approach used for Case 1 except look at increasing IA and/or *LID* globally, possibly increasing Shallow *n*, and possibly reducing *RTIMP*.

7.7.5.3 Case 3: Timing of hydrograph rising limb lags measured

The first things to look at are conveyance systems that are not being correctly represented in the model. Look at where the flow is concentrating and evaluate the topography. Are there channels smaller in width than the grid size lost due to elevation averaging? If these channels are doing most of the work, the conveyance needs to be included in the model using either *1D* channels or the multiple channel option. If this is not the case, evaluate the *n*-value assignments to these grids. They may need to be reduced. If none of these adjustments resolve the problem, try also reducing Shallow *n*.

7.7.5.4 Case 4: Timing of hydrograph rising limb precedes measured

The velocity in the main conveyance systems is too high. Try increasing *n*-values in the grids and channels that do the majority of the work. If none of these adjustments resolve the problem, try also increasing Shallow *n*.

7.7.5.5 Case 5: Shape of hydrograph does not match measured

For this case, there is the possibility that the rainfall data is inaccurate. Double check to make sure the rainfall distributions at grids containing a rain gage matches the measured rainfall reasonably well. After that, follow the procedures described for Cases 3 and 4 but focus on tributary areas rather than the entire watershed. Look for sub-watersheds where a change in timing could cause the discrepancies in hydrograph shape. Create floodplain hydrographs at the outlets of these sub-watersheds that can be plotted together to identify which areas might be causing the problem. Then apply the Case 3 or Case 4 approaches to those sub-watersheds.

7.7.6 Calibration: No Gage Data Available

The *FCDMC* standard approach when there is no gage data available for calibration is as follows:

1. Create a *HEC-1* model of the *FLO-2D* model domain. The purpose is to determine the rainfall excess volume that would remain on the surface if the *FCDMC* standard *1D* model hydrologic method were used. Typically, this can be done using a single watershed operation using the log-area-averaged *XKSAT* value and corresponding *DTHETA* and *PSIF* parameters. Use the same design rainfall that is applied to the *FLO-2D* design storm model. If there is significant variation in rainfall loss parameters within the study area, the use of multiple sub-watersheds may be necessary. The unit hydrograph parameters do not need to be actual values, just numbers that are not unreasonable. Check the *HEC*-1 rainfall excess volume against the total *FLO-2D* rainfall excess volume. The goal is to have as close a match as possible. Make adjustments globally to *LID* to accomplish the goal.
2. Perform the verification/confidence checks described in Section 7.7.3. If the available depth evidence is close to, but lower than, the *FLO-2D* model reported depths, consider the model adjustments completed. Otherwise, further adjustments to *LID* may be necessary. Consult with the *FCDM*C project manager before making further model adjustments.

7.8 Hydraulic Structures

From *FLO-2D Pro* Build 15.10.13 and newer, *FCDMC* recommends use of the general culvert equations option for most applications. Refer to Section 4.2.5.20. For builds prior to 15.10.13, the Rating Table or Rating Curve options should be used. Refer to *FLO-2D Hydraulic Structure Guidelines* (FLO-2D Software, Inc., 2014c) for guidance in applying the hydraulic structures component. The modeler should become very familiar with the REVISED_RATING_TABLES.OUT and the ERROR.CHK output files when using the Rating Table method. These files should be closely examined to verify that any automated rating table adjustments made at runtime are appropriate. The test models developed as a part of this verification project are available for study and can be used to help understand how the component works. Use of the General Culvert Equations for box culvert modeling is not recommended as of the date of this report. Instead, the rating table method should be applied until minor issues encountered during testing are resolved by the software developer.

7.9 Storm Drains

The *FLO-2D* storm drain component is recommended for use from *FLO-2D Pro* Build 15.10.13 forward. Application of the storm drain component requires a thorough understanding of the use of *EPA SWMM* 5 in addition to the *FLO-2D* interface with *SWMM*. Refer to the *FLO-2D Storm Drain Manual* (FLO-2D Software, Inc., 2015f) for guidance in using the storm drain component. The test models developed as a part of this verification project are available for study and can be used to help understand how the component works.

7.10 Floodplain Delineation Method and Procedures

7.10.1 General

The use of *FLO-2D* results for delineation of floodplain boundaries can be divided into two classifications: 1) confined flow and 2) unconfined flow. These classifications are addressed separately in the following sections.

7.10.2 Confined Flow

When the system modeled has higher ground at the edge of the floodplain that confines the flow, the following procedure can be used to define the floodplain boundary using *ArcGIS*:

1. Create a GIS point feature class of the center of each *FLO-2D* grid element. The database table should contain the grid element number, grid element ground elevation, maximum flow depth, and maximum *WSEL*.
2. If the model computes rainfall losses, then revise the *WSEL* of the grid elements that have very small flow depths that represent shallow flow outside of the desired floodplain. Depths of 0.1 feet and lower usually suffice. Select those grid elements and set the *WSEL* value to be the ground elevation. Do the same for any tributary inflow washes that do not warrant a defined floodplain.
3. Create a triangulated *TIN* surface of the maximum *WSEL*s.
4. Create a *TIN* of the ground surface.
5. Use the *ArcGIS 3D* Analyst Surface Difference tool to create polygons of areas above, below and at the same elevation.
6. Export the ‘above’ polygons and then convert them to a line feature class. Perform edits as needed to obtain the final floodplain boundary.

7.10.3 Unconfined Flow

Unconfined flow for this procedure includes large areas of relatively shallow sheet flow and/or distributary flow, excluding active alluvial fan areas. These unconfined flow areas are problematic for defining a regulatory floodplain. Flow is constantly breaking out or diverging along the floodplain fringe and then decreasing as it moves downstream by spreading out and infiltrating. The issue is deciding where to draw the line that defines what is to be regulated under the Floodplain and Drainage Regulations, and what should be addressed only through the Drainage Regulation. The following is the *FCDMC* approach to accomplish this:

1. The *FCDMC* Floodplain Regulations allow regulating areas with a 100-year peak discharge of 50 cfs or greater. Define flow splits that have a total peak flow rate of 50 cfs or less.
2. Check the flow depths in the flood fringe breakout area with a flow rate of 50 cfs or less. If the flow depth is greater than 1-foot, consider leaving those grids in the defined floodplain. If the flow depths immediately downstream drop below 1-foot as flow spreads and dissipates, use judgment in drawing the floodplain boundary.
3. Use the grid boundary to draw the floodplain boundary. There is no way to define a catch point with higher ground for these circumstances so using the grid boundary shows the basis for the delineation.
4. For breakouts with a 100-year peak discharge of greater than 50 cfs, continue to delineate downstream until the flow rate drops below 50 cfs.

This is a manual time consuming process. An example is shown on Figure 7.2. The floodplain boundary is the thick blue line. The labels are maximum *WSEL*, discharge and flow direction, and maximum depth. The total peak discharge in the breakout is about 23 cfs and the flow depth drops to below 1-foot, so the breakout to the southeast was not included in the floodplain.

7.10.4 Floodway Delineation

From the *FCDMC* perspective, there is currently no acceptable way to define a floodway that meets the *FEMA* standard definition using *FLO-2D*. *FCDMC* is working on a solution to this problem, but nothing is available at this time. Since the majority of flood hazards where *FLO-2D* should be applied consist of shallow flow, *FCDMC* has adopted the following approach in the past for the Rio Verde *ADMP* (FCDMC, 2007):

1. Define an AE Zone floodplain using the detailed *FLO-2D* model maximum *WSEL* results and the procedures set forth in Section 7.10.
2. Enforce a zero rise floodplain policy for the floodplain areas. Development on a lot must have a drainage design report that shows that the improvements do not increase the 100-year *WSEL* above

the base flood elevation, and that drainage patterns, depths and velocities match predevelopment conditions on the downstream side(s) of the lot.

3. The use of *HEC-RAS* is allowed for the hydraulic analysis in support of development of the parcel, or a smaller grid *FLO-2D* model may be used. The total peak discharges from the *FLO-2D* model results entering and leaving the parcel are used in the analysis.

7.11 Model Limitations

7.11.1 Number of Grids

Currently, the maximum practical limit is about 2 million grid elements. *FCDMC* prefers to keep model size under 1.5 million grids by creating multiple adjoining models. *FLO-2D* has an option to automatically create an INFLOW.DAT file from outflow cell hydrographs, which is used to ease setup of adjacent models. The *FLO-2D GDS* program can be used to create the grid and compute average elevations for large models but many of the *GDS* tools will not function properly for very large numbers of grid elements so *FCDMC* recommends use of *GIS* tools for development of most other input data files.

7.11.2 Grid Size

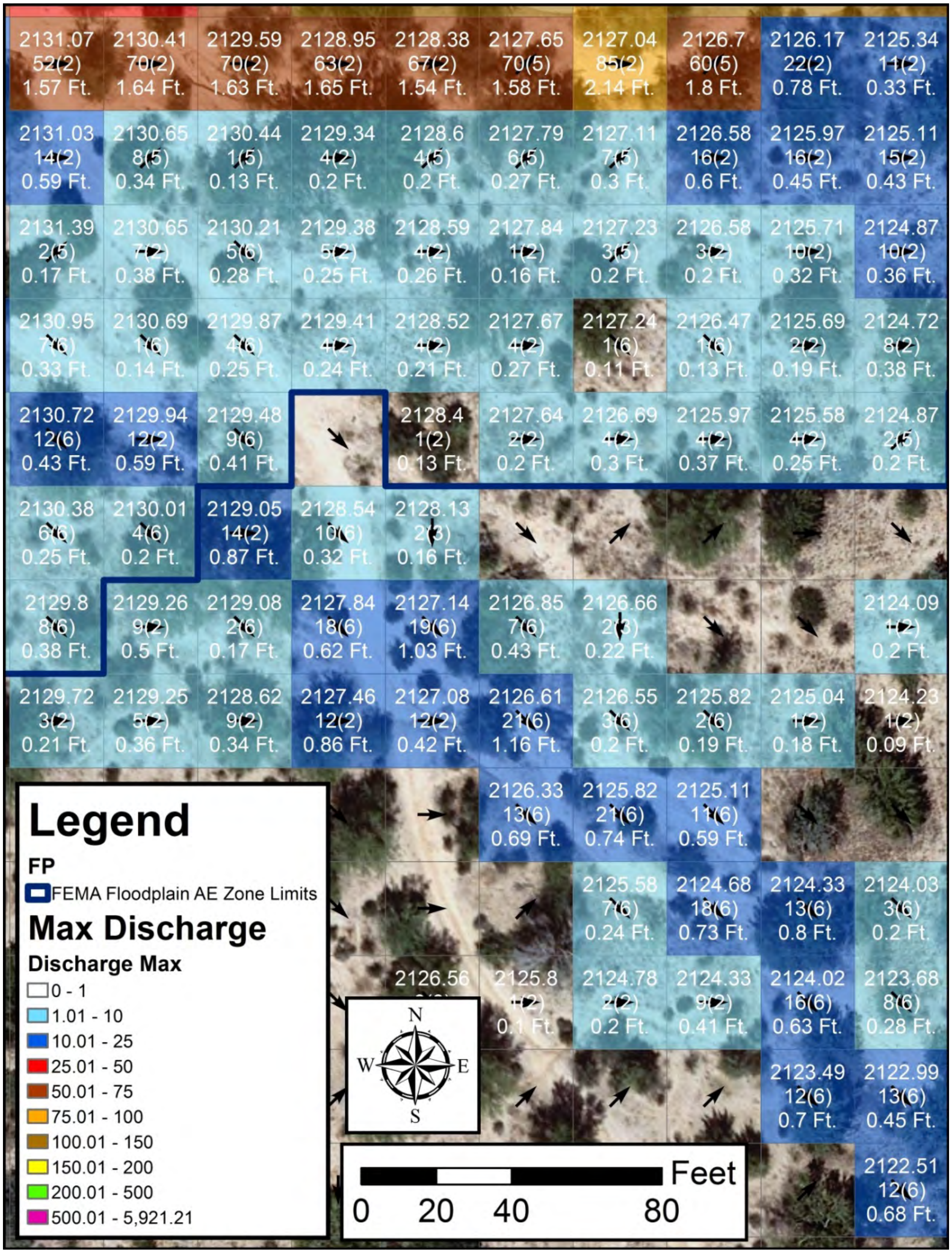
7.11.2.1 Hydrologic Modeling

The *FCDMC* recommended maximum grid size for a strictly hydrologic *FLO-2D* model is 50-feet. *FEMA* may require a 10-meter maximum size, so always check with the reviewing agency first. Depending on the nature of the terrain, the multiple channel and/or *1D* channel components may need to be applied in order to accurately simulate watershed response time.

7.11.2.2 Hydraulic Modeling

FCDMC has experimented with and applied 50-, 35-, 30-, 25-, 20-, 15- and 10-foot grid sizes for use on large-scale watershed studies where the floodplain grids are relied on for the majority of the hydraulic computations. As a result of these trials, *FCDMC* recommends a maximum grid size of 25-feet and has been using a 15- or 20-foot grid size for most recent models. Grid size selection is dependent on the study goals, level of detail desired, the maximum number of grid elements, and the accuracy of the available topographic mapping. *FCDMC* is now performing complex urban *2D* modeling using a grid size of 15-feet. A grid size of no smaller than 10-feet is recommended as the very small time steps required for such small grid elements may exceed the model technical limits and model run times may be excessively long. Larger grid sizes could be used for riverine system models where the floodplain grids are used to model wide uniform overbank flows in combination with the *1D* channel component.

Figure 7.2 Unconfined Flow Floodplain Boundary Example



7.11.2.3 Discharge Flux Considerations

Per the *FLO-2D* Input Data Manual (FLO-2D Software, Inc., 2015d), the following criteria is helpful when selecting a model grid size:

$Q_{peak}/A_{surf} < 10.0$ cfs/ft^2

The closer Q_{peak}/A_{surf} is to 3.0 cfs/ft^2, the faster the model will run. If the Q_{peak}/A_{surf} is much greater than 10.0 cfs/ft^2, the model will run more slowly. This relationship is particularly important when large flow rates are conveyed through the grid system without dispersal. This situation should be considered when establishing the grid element size.

This relationship is also important when adding inflow hydrographs to the system. A common practice is to assign an inflow hydrograph to only one grid element. Quite often, this approach violates the above relationship and slows the model down. To address this, the inflow hydrograph should be divided into multiple hydrographs and spread out over multiple grid elements. Care should be taken to use the grids that represent the floodplain the inflow applies to and to account for available conveyance when dividing the inflow hydrograph.

7.11.3 Topographic Mapping

The basis for any hydraulic model is the topography. This is particularly true for the *FLO-2D* model since the grid element representation is essentially a *DEM*. The accuracy of the topographic mapping must be commensurate with the goals for application of the *2D* model and of sufficient detail to produce an accurate *DEM* of the grid size. When the model area contains linear features such as levees, roadway embankments, basins, streets with curbs and gutters, and a complex channel system, the makeup of the underlying point data used to produce the topographic surface model becomes very important. A limitation of the *FLO-2D* model is that the grid size may make it difficult to accurately account for these linear features. *LIDAR* datasets, which is often a cost effective method for obtaining topographic data, although very detailed, often lack critical details needed to simulate such features in a time or cost effective manner. *FCDMC* prefers a traditional photogrammetric mapping approach when these types of features have to be accurately represented in the model. This approach is preferable in order to obtain *3D* break lines that can be used to hard-code the effects of these features into the model. It is also a more cost effective approach to obtain cartographic features used to simulate impervious area and flow obstructions such as building and walls.

The *FCDMC* has topographic and cartographic mapping specifications that define how this information is to be prepared and supplied in order to facilitate *2D* model development. These specifications are available from the *FCDMC* Engineering Division.

7.11.4 *1D* Channels

7.11.4.1 General

The *FLO-2D 1D* channel component is recommended for use with all builds of *FLO-2D* 2009.06 and *FLO-2D Pro*. Application of the storm drain component requires a thorough understanding of the use of *1D* modeling techniques in addition to the *FLO-2D* interface. Please refer to *Channel Guidelines* (FLO-2D Software, Inc., 2015b) and *Channel Termination Guidelines* (FLO-2D Software, Inc., 2015c) for guidance in using the *1D* channel component. The test models developed as a part of this verification project are available for study and can be used to help understand how the component works.

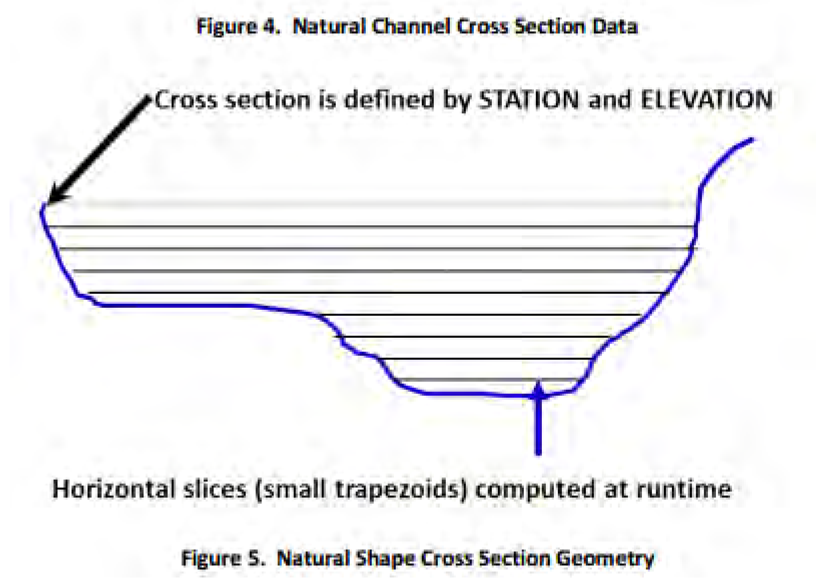
The following sections provide insight and guidance on several aspects of *1D* channel modeling within *FLO-2D*.

7.11.4.2 Natural Channel Cross Sections

Two important aspects of how the *FLO-2D 1D* channel component is implemented are discussed in this section. The first relates to how *FLO-2D* preprocesses and applies the cross section data. When the data files are read, a rating table of hydraulic parameters are created for every cross section, calculated using horizontal slices to form small incremental depth trapezoids as shown in Figure 7.3. It is important that the appropriate number of points is used to define each cross section. There should be a minimum of 8 points used for natural cross sections, even and especially if a prismatic rectangular or trapezoidal cross section is being defined. The two bottom corners need an additional point on each side close to the two bottom corners in order for the horizontal slices to be established correctly. To avoid problems because of too few points used, it is recommended that the prismatic channel option be used for rectangular and trapezoidal channels rather the natural channel option.

It is not recommended to extract an actual cross section from the topographic surface for every grid element. Instead, extract cross sections at the same spacing as is normally used for a *HEC-RAS* model and then interpolate cross sections for the intermediate grid elements. The cross sections should be setup and tailored to meet the goals of the model. If the model is for a large peak discharge such as the 100-year storm, care should be taken to remove unnecessary points and small fluctuations that do not affect the overall conveyance at peak. There should not be dramatic changes in cross sectional area and the slope should be representative of the reach with ineffective flow areas, both along the bottom and sides, removed. Each cross section should represent the main conveyance channel not the entire floodplain.

Figure 7.3 *FLO-2D* Natural Channel Parameter Rating Table

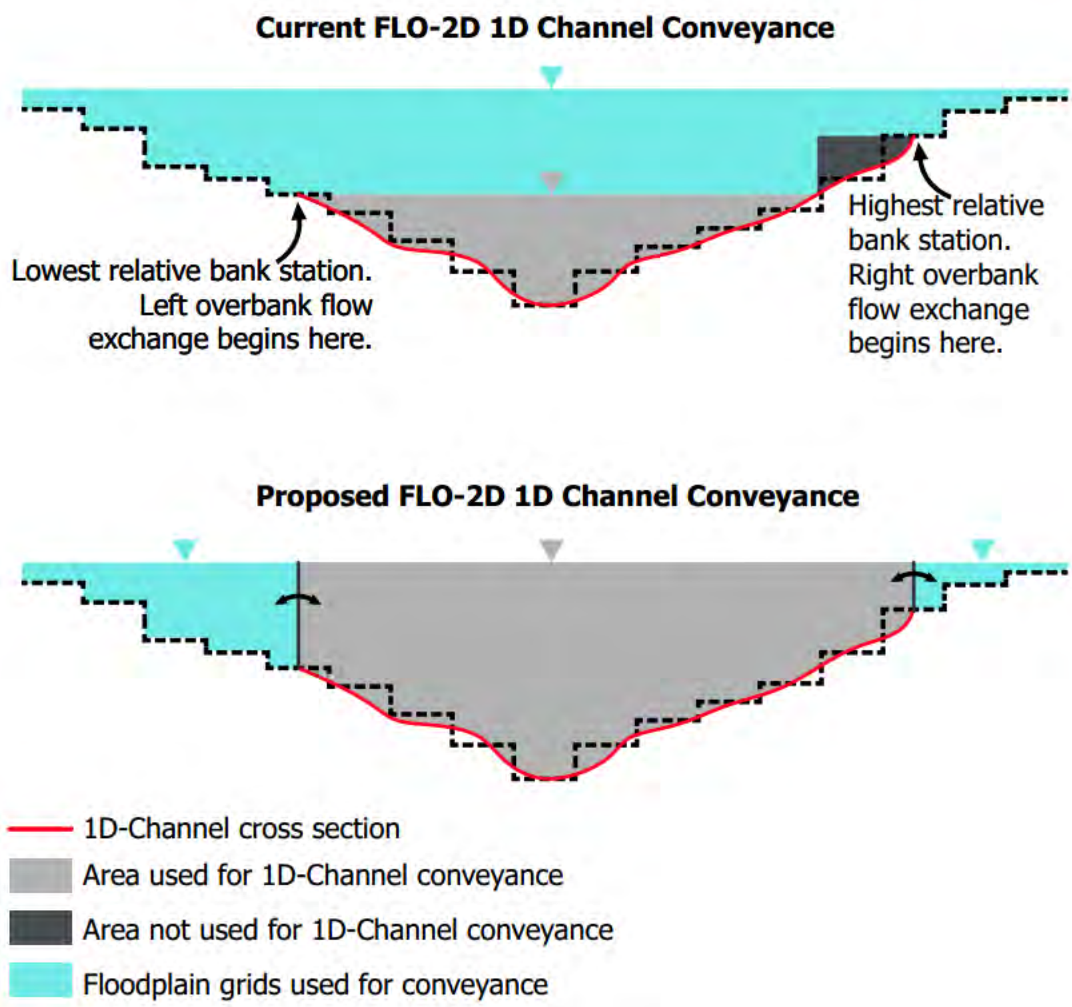


The current *FLO-2D 1D* channel routine computes the cross section area for conveyance in a manner different than *HEC-RAS*. Refer to Figure 7.4. *FLO-2D* locates the lowest relative high bank and uses the elevation to stop the development of the hydraulic parameters rating table data. *HEC-RAS* conversely uses all available area by extending the low side vertically. The *FLO-2D* approach can result in some cross sectional area not being accounted for properly. An example of that area is shaded in dark grey on Figure 7.4 on the cross section labeled "Current *FLO-2D 1D* Channel Conveyance." *FCDMC* is currently working with *FLO-2D* Software, Inc. to revise this approach to more closely match *HEC-RAS*. The revisions will be designed to simulate the cross sectional area scheme labeled "Proposed *FLO-2D 1D* Channel Conveyance." Check the *FLO-2D* updates description file periodically to see if this change has been made. Until then, the modeler should setup the cross sections so that this problem does not occur. Essentially, the left and right banks should be close to the same elevation.

7.11.4.3 Use of Floodplain Grids

The use of only floodplain grids to model *1D* channels, particularly relatively deep confined channels, should be undertaken cautiously. Confined channel grids do not use the entire grid width to convey flow. The floodplain grid hydraulics also only uses the grid base width for wetted perimeter. Refer to Section 4.2.2 for more detail. These factors can have an impact on the results, typically slightly increasing the flow depth and velocity. If more detailed hydraulics of the channel is needed, the *1D* channel component should be applied instead.

Figure 7.4 *FLO-2D* Cross Section Area Used for Conveyance



7.11.5 Multiple Channels

The multiple channel component should be applied only when the grid does not adequately model the effects of existing multiple shallow inset natural channels that are expected to increase in width during a major flow event. If used to model one or more small channels that are not expected to expand, caution should be applied if the flow depth is expected to exceed the inset channel capacity. For these cases, *FLO-2D* will not switch to the remaining overbank area within the grid element. Instead, it will keep increasing the depth in the channel by extending the walls vertically. Also, the multiple channel component is based on the wetted perimeter only including the channel base width, not the sides. When flow depths exceed two (2) feet, the error associated with this assumption increases and the results may not be appropriate for use. Instead, use of the *1D* channel component should be considered.

7.12 Application of Model Results

7.12.1 Design Applications

Before using the *FLO-2D* model results from an *ADMS*, *ADMP* or other *FCDMC* study, carefully review the basis for the models described in the *TDN,* all assumptions made, and recommendation on use of the results. Additional refinement of the model may be needed in a particular area for design purposes. The user should keep in mind that the *ADMS*/*ADMP* models are intended for flood hazard definition along major flow areas. Depths and discharge rates in areas with small contributing watershed may need careful review to determine if the results are reasonable for the intended application.

7.12.2 Assignment of Base Flood Elevation at a Building

The goal is to determine the highest *WSEL* at the building. This determination should exclude interior building grids that are completely blocked by *ARF*, and adjacent grids that are not a part of the floodplain such as grids representing high adjacent cut banks or behind retaining walls where the *WSEL* only represents local runoff. A recommended process for assignment of the Base Flood Elevation (*BFE*) at a building is as follows:

1. Prepare a map of the *FLO-2D* model results that includes flow direction arrows, the maximum *WSEL*, and the peak discharge at each grid.
2. Case1: The ground slope is steep (fall generally greater than 2 feet from the upstream side to the downstream side of the building) and the structure is built on an elevated pad where flow ponds against, and flows around, the upstream side of the structure. For this case, use either the highest grid *WSEL* on the upstream side, or the average *WSEL* of the grid elements along the upstream side for the *BFE*, based on engineering judgment.
3. Case 2: The ground has a relatively flat slope (fall from the upstream side to the downstream side is 1 to 2 foot or less). For this case, compute or plot the water surface profile along the structure parallel with the direction of flow, assuming the *WSEL* is at the center of each grid element. Interpolate a *WSEL* at the upstream side of the building and assign as the *BFE*.

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A. DIGITAL DATA

The supporting digital data is available for review but not included in this document. The digital files include several hundred gigabytes of data that can be provided on an external hard drive. Coordinate with the Special Projects Branch, Engineering Division, *FCDMC* to obtain a copy of this information, or subsets of the information.

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