Guidance for 2-Dimensional Model Development in Riverine Systems

Prepared for

Urban Drainage and Flood Control District

April 2012

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Acronyms and Abbreviations

1-D	One Dimensional
2-D	Two Dimensional
3-D	Three Dimensional
DFIRM	Digital Flood Insurance Rate Map
EAP	Emergency Action Plan
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
FHAD	Flood Hazard Area Delineation
HEC	Hydraulic Engineering Center
RAS	River Analysis System
SWMM	Storm Water Management Model
UDFCD	Urban Drainage and Flood Control District
USACE	United States Army Corps of Engineers

1 Introduction

1.1 Background

Riverine hydraulic modeling within the United States has been largely dominated by the use of one-dimensional (1-D) analysis programs, such as the U.S. Army Corps of Engineers (USACE) HEC-RAS River Analysis System software or the Environmental Protection Agencies (EPA) Storm Water Management Model (SWMM). With the advances in remote sensing technology that provide detailed and accurate terrain models, hydraulic analyses and floodplain delineations have become more correct in depicting the flow conditions through riverine systems. This includes improved understanding and identification of complex flow splits with multiple flow directions.

1-D models have been traditionally used in the US to analyze riverine systems, however these models are limited in analyzing complex flow systems due to the fundamental assumption that flow is in a single direction. With the increased computational power that is now available to Engineers and Regulators, the use of two-dimensional (2-D) hydraulic models can be quickly developed to gain a better understanding of the complex flow, flow direction and flow volume for riverine systems.

Current regulations for floodplain modeling are generally geared toward 1-D modeling with kinematic wave routing assumptions. Many flooding scenarios are better represented by 2-D models than 1-D models; in particular areas of split flows caused by hydraulic structures, urban flooding areas and alluvial fan analysis. The purpose of this paper is to provide guidance on when 2-D models should be utilized and how to correctly develop a 2-D model for riverine systems. In light of the current regulatory environment, most legacy models are 1-D and many regulations surrounding floodplain development and floodplain management are geared toward 1-D modeling results. This paper will also provide guidance on how the results of a 2-D hydraulic model can be used to quickly and efficiently develop 1-D models acceptable to regulatory agencies and municipalities for riverine systems.

1.2 Regulatory Issues

The regulatory environment for floodplain development and regulation is currently based upon the use of 1-D models; though there have been a few examples of 2-D models incorporated into the Federal Emergency Management Agency's (FEMA's) Digital Flood Insurance Rate Map (DFIRM) Products. The primary challenge that comes with using 2-D models for floodplain management is reconciling flow rates computed from the dynamic wave analysis used in a 2-D model with adjacent models developed using kinematic wave routing and the traditional steady-state flow condition assumptions employed in a 1-D model. There are also challenges associated with the availability of the data, software and computational power to efficiently and cost-effectively share and update 2-D models when development or other changes occur within a floodplain. It is because of these challenges that this study was initiated to develop an approach for utilizing the hydraulic analysis produced from a 2-D model to create and/or validate a traditional 1-D model that can be utilized by the engineering and regulatory communities.

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2 Comparison of 1-D Models and 2-D Models

2.1 2-D Modeling Overview and Comparison to 1-D Modeling

There are two major modeling methodologies used to delineate floodplains: 1-D steady state models and 2-D models. These models compute water surface elevations, velocities and floodplain extents by making different assumptions about how flow propagates. In general, both 1-D and 2-D models solve the Saint-Venant equations with some underlying assumptions to simplify the process. In 1-D flow programs such as HEC-RAS or EPA-SWMM, the flow direction is assumed to be in the downstream direction (Figure 2-1) and is not explicitly computed, while in 2-D models, the flow directions are explicitly computed in any of eight separate directions (Figure 2-2). In addition to flow direction, 2-D models compute flow velocity and flow rate in any of the eight directions.

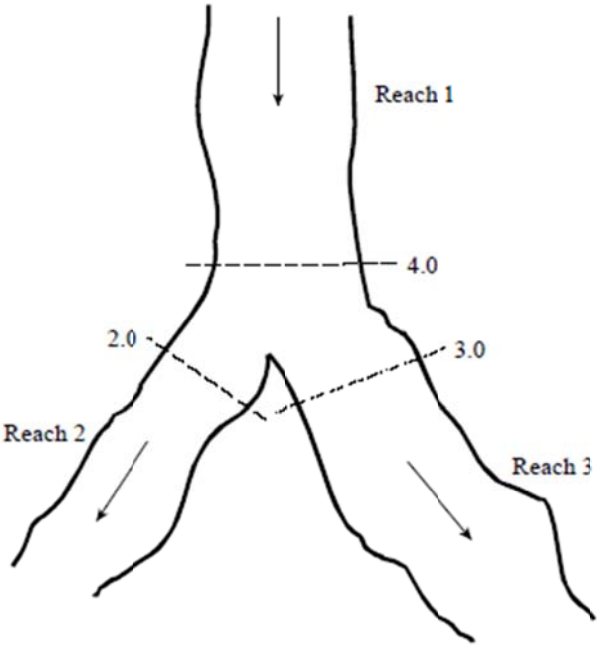

Figure 2-1 - Assumed Flow Direction for 1-D Models

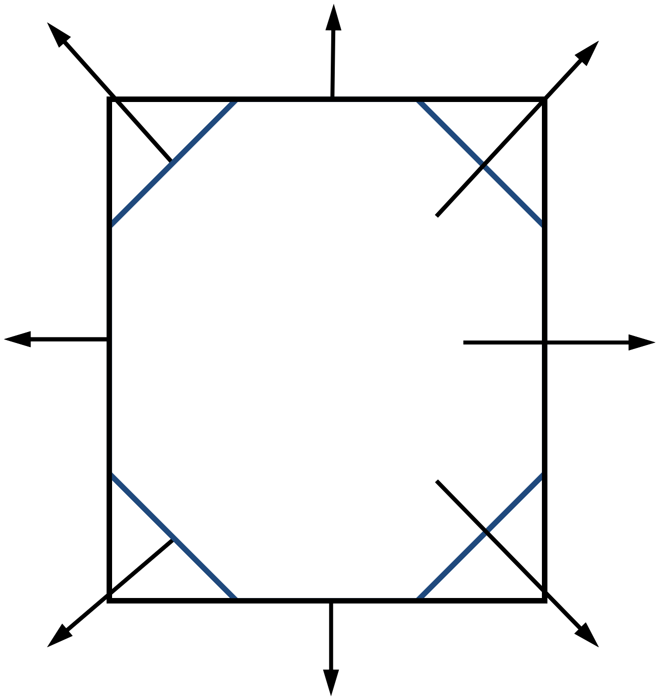

Figure 2-2 - Computed Flow Directions for 2-D Models

For a 1-D model, geometric data and the physical properties of the stream system are computed utilizing cross sections and channel profiles while a 2-D model utilizes grid cells with elevation and roughness data to represent the ground surface elevation. To develop the grid for a 2-D model, terrain data is required to be rasterized. This interpolation method aggregates multiple elevations points across a grid cell into a single elevation point that becomes the assigned elevation for the entire grid cell. The rasterization process can subtly change the shape of a cross section because of the assimilation of multiple elevations points into one grid cell. Figure 2-3 depicts the difference between a cross section cut from a set of contour data for a 1-D model and one cut on a grid for a 2-D model.

In a 2-D model, flow direction may change with each time step, requiring the program to come up with a numerical solution to the Saint-Venant equations at each grid cell and for each time step. Therefore, 2-D models must complete a large number of calculations, which can result in excessively long model run times and model instability. Recommended approaches for developing a stable 2-D model to reduce run time while maintaining meaningful results are discussed in detail in Section 4, 2-D Modeling Keys to Success.

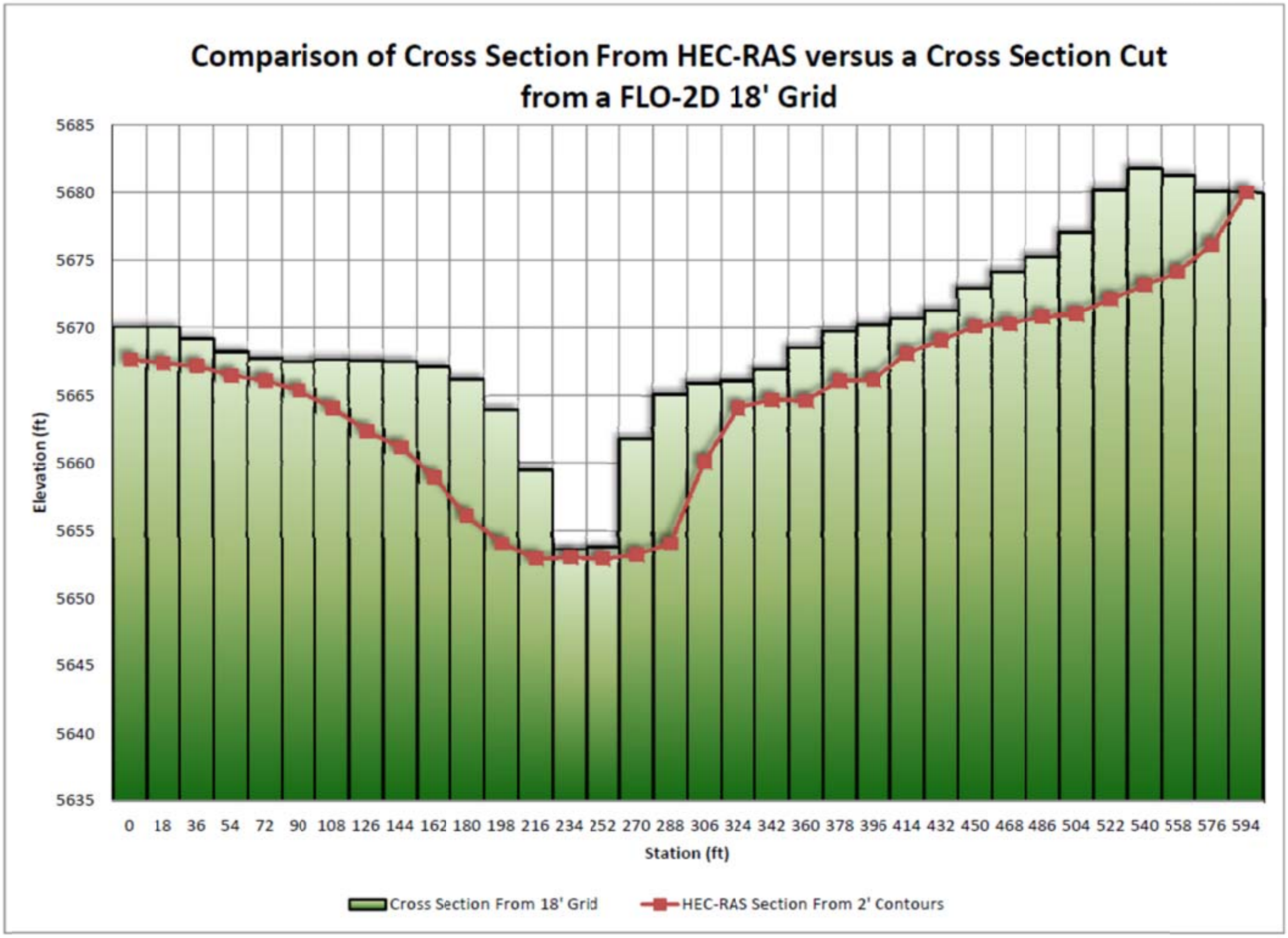

Figure 2-3 - Comparison of a HEC-RAS Cross-Section and Rasterized Cross-Section

In addition to the differences regarding flow direction, there are other major computational differences between the two types of models. Transverse velocity and momentum and transverse variations in water surface depths are handled differently in 1-D and 2-D models. In 1-D models, transverse velocity, momentum and water surfaces depths are not calculated and are assumed constant across the entire cross section. For 2-D models, these variables are explicitly calculated for each grid cell. The calculation of both flow depth and velocity provide in-depth information about the behavior of flow in the floodplain. This information can also be used to help understand the direction and magnitude of flow at any point within the floodplain.

2.2 Comparison of Kinematic and Dynamic Wave Hydrograph Routing

One of the major differences between 1-D steady state models and 2-D models is the methodology by which they route the peak flow hydrographs through the model domain. 1-D steady state models use kinematic wave routing theory to route flows downstream while 2-D models use dynamic flood routing.

Kinematic wave routing assumes that inertial and pressure forces are negligible in the Saint-Venant equations. The theory assumes that the weight of the water flowing downstream is approximately balanced by the resistive forces of channel bed friction. These assumptions then dictate that flood flows moving in the downstream direction will not accelerate appreciably and the flow will remain relatively uniform, defining the kinematic wave propagation. In addition, due to the assumptions made with kinematic flow, backwater effects and storage are generally considered negligible. In contrast, dynamic wave routing assumes that inertial and pressure forces are not negligible, and that backwater effects and storage can affect the wave propagation. This results in dynamic wave controlling the propagation of long waves in shallow water.

Studies reviewed by the USACE have drawn the conclusion that kinematic waves will ultimately dominate the flow characteristics occurring for overland flows and small watershed channel flows when the Froude number is less than 2. As a result, flood flows for watershed systems like those within the Denver metro area are generally dominated by kinematic waves because the passage of the flood wave appears as a uniform rise and fall in the water surface elevation over a relatively long period of time. This is the reason why flood studies are completed with kinematic wave routing. An additional reason for utilizing kinematic wave routing is the consideration of backwater and storage effects. Many regulations require that areas of storage be owned and maintained by local governments in order to assure the storage benefits will remain in perpetuity. Because backwater effects behind roads and overbank storage in the floodplain is not protected by regulation, removing the effects from routing equations provides a conservative estimate of peak flow rates that can be used to plan for future infrastructure improvements.

Three case studies were evaluated for the purpose of this study. These case studies are discussed in detail in Section 3 1-D and 2-D Case Studies. The three case studies were:

1. Willow Creek located in north Douglas County, City of Lone Tree, City of Centennial and Arapahoe County;
2. Coal Creek located southwest of the Town of Superior; and
3. Little Dry Creek downstream of the Englewood and Holly Dams.

The Willow Creek watershed routing results using the traditional 1-D modeling methodology of kinematic wave assumptions was compared to the routing results using the dynamic wave approach.

For the Willow Creek model traditional 1-D analysis:

- CUHP and EPA SWMM models were used to develop original hydrology
- SWMM was calibrated to previous published results
- Kinematic routing was utilized

For the Willow Creek 2-D routing analysis:

- CUHP hydrographs were used for direct inflow basins
- Design point hydrographs taken from SWMM output were used for tributary streams
- Dynamic routing was utilized

Results from the analysis can be found in Figure 2-4, Peak Flow Comparison for 1-D and 2-D Models. Overall, it was found that peak flow rates were reduced by 32% on average when run through the 2-D model. This was to be expected as backwater and storage were calculated in the 2-D model that had the ultimate effect of reducing the flow rate. However, the flow volumes compared between the two models were within 3% of each other. The conservation of volume indicates that both models are computing flows correctly, but the difference in peak flow rates need to be taken into consideration when creating a 1-D model from 2-D model results. This will be discussed further in Section 5, creating 1-D Models from 2-D Results.

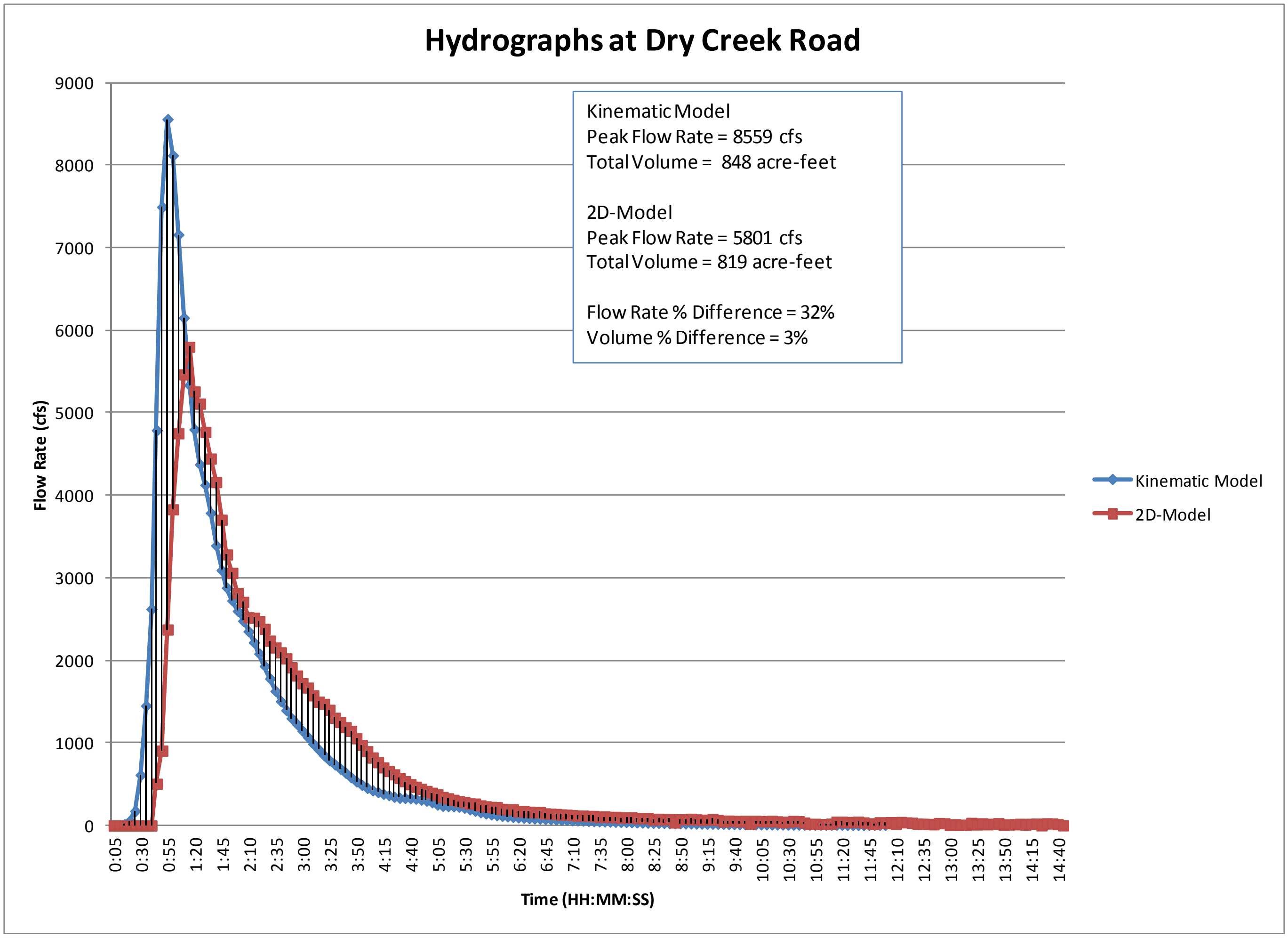

Figure 2-4 - Peak Flow Comparison for 1-D and 2-D Models

2.3 Basic 2-D Hydraulic Theory

2-D models are generally simple volume and momentum conservation models. This means that a volume of water defined by the inflow hydrographs is moved around the model domain. This means that the sum of the outflow volume off the grid domain, storage volume on the grid, and losses needs to be close to equaling the volume of the inflow hydrograph for a successful model. Volume conservation becomes an important review tool for 2-D modeling because it becomes an indication of numerical stability and accuracy.

2-D models generally are governed by the continuity equation and dynamic wave momentum equation. These two equations are solved generally using a central finite difference scheme. This process models the flood wave progression over time through the model domain. The hydraulic calculations are primarily impacted by topography and the resistance of each grid cell element to flow as defined by Manning's *n* Value.

2.4 2-D Additional Data Requirements

2-D modeling requires additional information that is not needed for a traditional 1-D model. Although the information to develop a 2-D model is closely related to the data required for 1-D modeling, the amount of data is more extensive. Terrain data for 1-D models can be as simple as spot elevations for surveyed cross sections. For 2-D models, a continuous terrain model is required for the entire modeling domain. The terrain data needs to be surveyed in sufficient detail and the grid size selected to ensure adequate detail and flow resolution can be obtained. An example of a 14-foot grid cell 2-D bathymetric grid is represented in Figure 2-5.

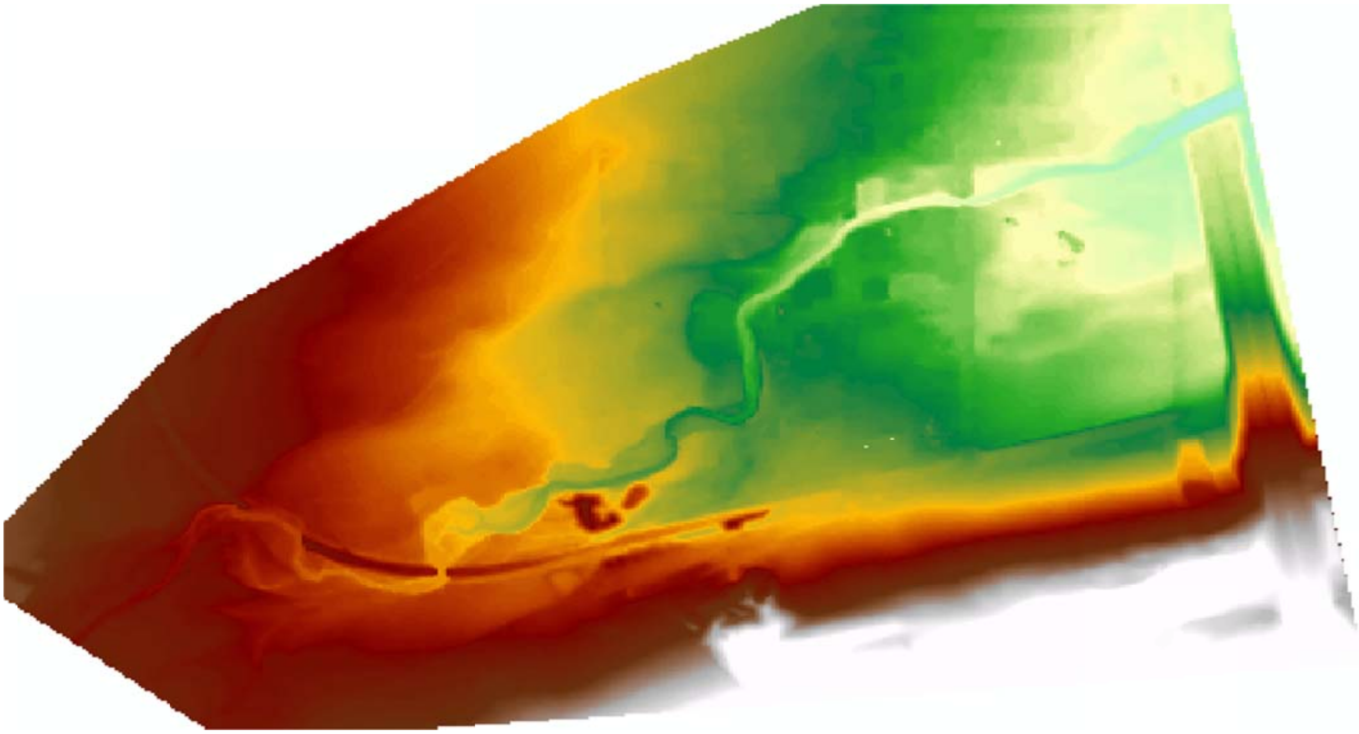

Figure 2-5 - Example Bathymetric Terrain Data

For a 2-D model, Manning's *n* roughness (*n*) values also need to be determined for the entire modeling domain rather than just along the drainage corridor as for a 1-D model. To determine Manning's *n* values for the entire modeling domain, aerial photographs, zoning maps and land use maps are employed to spatially represent land use conditions that can be related to Manning's *n* values for the entire modeling domain. Figure 2-6 contains land use data that was used to define Manning's *n* values for the Coal Creek case study. A broader discussion of how to assign Manning's *n* values will be discussed in Section 4, 2-D Modeling Keys to Success.

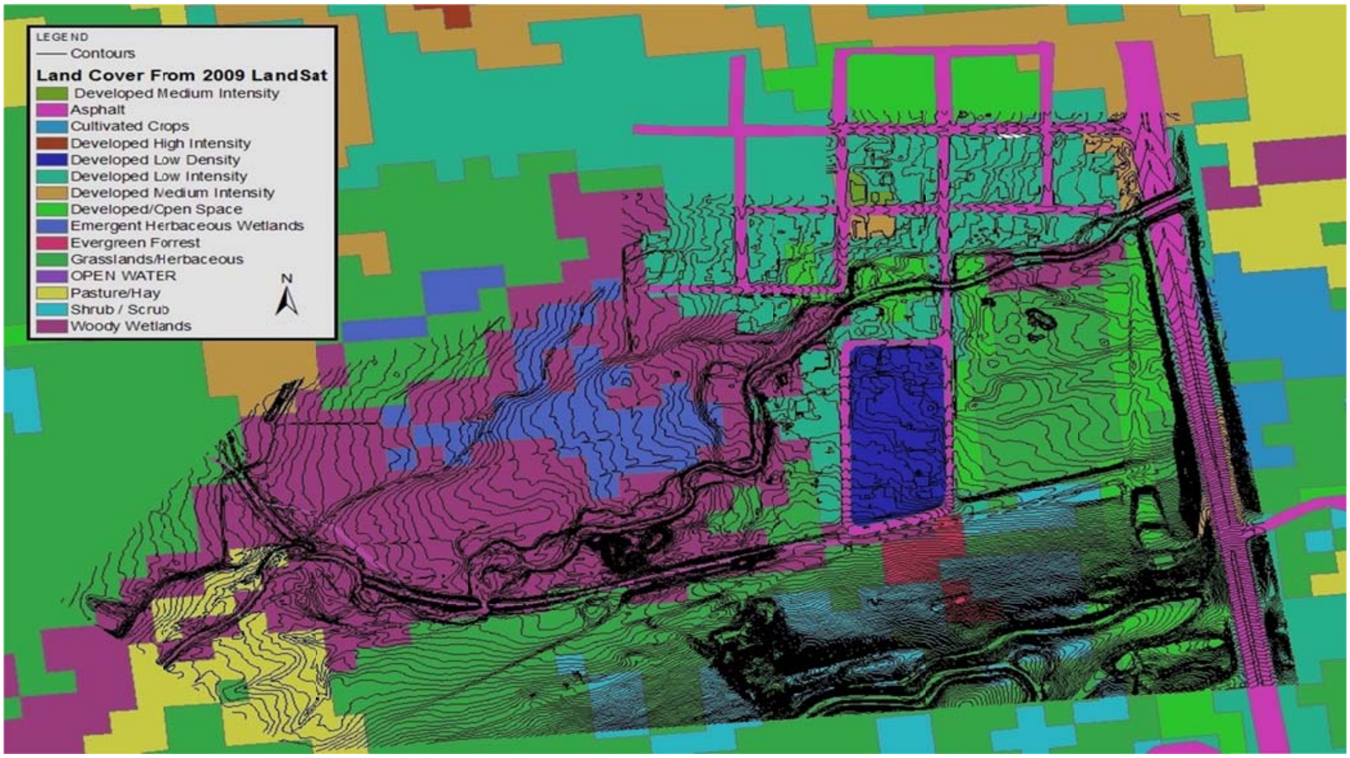

Figure 2-6 - Example Land Use Data for determination of Manning's *n* values

The final set of data required for a 2-D model is inflow hydrographs. 1-D regulatory models use steady state peak flows and do not have the time variable hydrographs needed for input into the 2-D modeling domain. Hydrograph loading in a 2-D model requires the user to select the most appropriate location on the grid to load the flow. A broader discussion of development of 2-D models is presented in Section 4, 2-D Modeling Keys to Success.

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3 Comparison of 1-D Models and 2-D Models

Three case studies were created in order to develop guidance and recommendations for developing 1-D regulatory models from 2-D hydraulic models for varying floodplain situations. The three case studies were:

4. Willow Creek located in north Douglas County, City of Lone Tree, City of Centennial and Arapahoe County;
5. Coal Creek located southwest of the Town of Superior; and
6. Little Dry Creek downstream of the Englewood and Holly Dams.

3.1 Willow Creek

Willow Creek is a channel located south of the Denver metro area and travels through fully developed portions of northern unincorporated Douglas County, City of Lone Tree, City of Centennial, City of Greenwood Village and unincorporated Arapahoe County. Due to its overall riverine nature, this channel was chosen as the baseline condition to determine the efficiency of modeling riverine channels using 2-D models, including model set up time, model run time and accuracy of the results of routed hydrographs and floodplain delineation. The Willow Creek watershed was an ideal candidate for a case study because the floodplain was recently mapped using HEC-RAS along with well-documented hydrology using the Colorado Urban Hydrograph Procedure (CUHP) and EPA-SWMM 5.0, which could be used to compare routing results to FLO-2D. Figure 3-1 depicts a portion of the Willow Creek Flood Hazard Area Delineation (FHAD) created in 2011.

Figure 3-1 - Example of the Willow Creek 1-D HEC-RAS FHAD

Table 3-1 contains details of the Willow Creek watershed.

TABLE 3-1
Watershed Details for Willow Creek

Drainage Area (acres)	**Average Watershed Slope (%)**	**Typical Development Characteristics**	**Length of Study Reach (miles)**	**Peak Flow Rate (Q) (cfs)**
6,047	0.80	Fully Developed with a mixture of residential and commercial land use	5	9,000

Table 3-2 includes the pertinent characteristics of the 2-D model that affected the model performance and resolution. Model development times for this case study were extensive, with most of the time being attributed to manually entering 30 hydrographs and developing the rating curves required for inputting the numerous hydraulic structures into the model.

TABLE 3-2
2-D Model Parameters for the Willow Creek Watershed

Grid Size (ft)	Number of Grid Cells	Number of Hydraulic Structures	Model Run Time (hours)	Model Development Effort
18	169,000	10	14	Extensive

3.2 Coal Creek

The Coal Creek watershed is located northwest of the Denver metro area with the Coal Creek study reach lying southwest of the Town of Superior. The case study reach is approximately 1 mile in length and was included due to the multiple split flows that occur within the floodplain. Coal Creek was originally modeled in HEC-RAS utilizing a trial and error method that took over two weeks to complete. Flow splits were determined by using lateral weirs to determine the split flow rates and flow paths (Figure 3-2).

The 2-D model developed for the Coal Creek Study was created quickly with reduced effort and provided greater insight into the split flow rates and flow paths than the assumptions made in the 1-D model. This model was used to create procedures and guidelines for creating and validating 1-D models using a 2-D analysis.

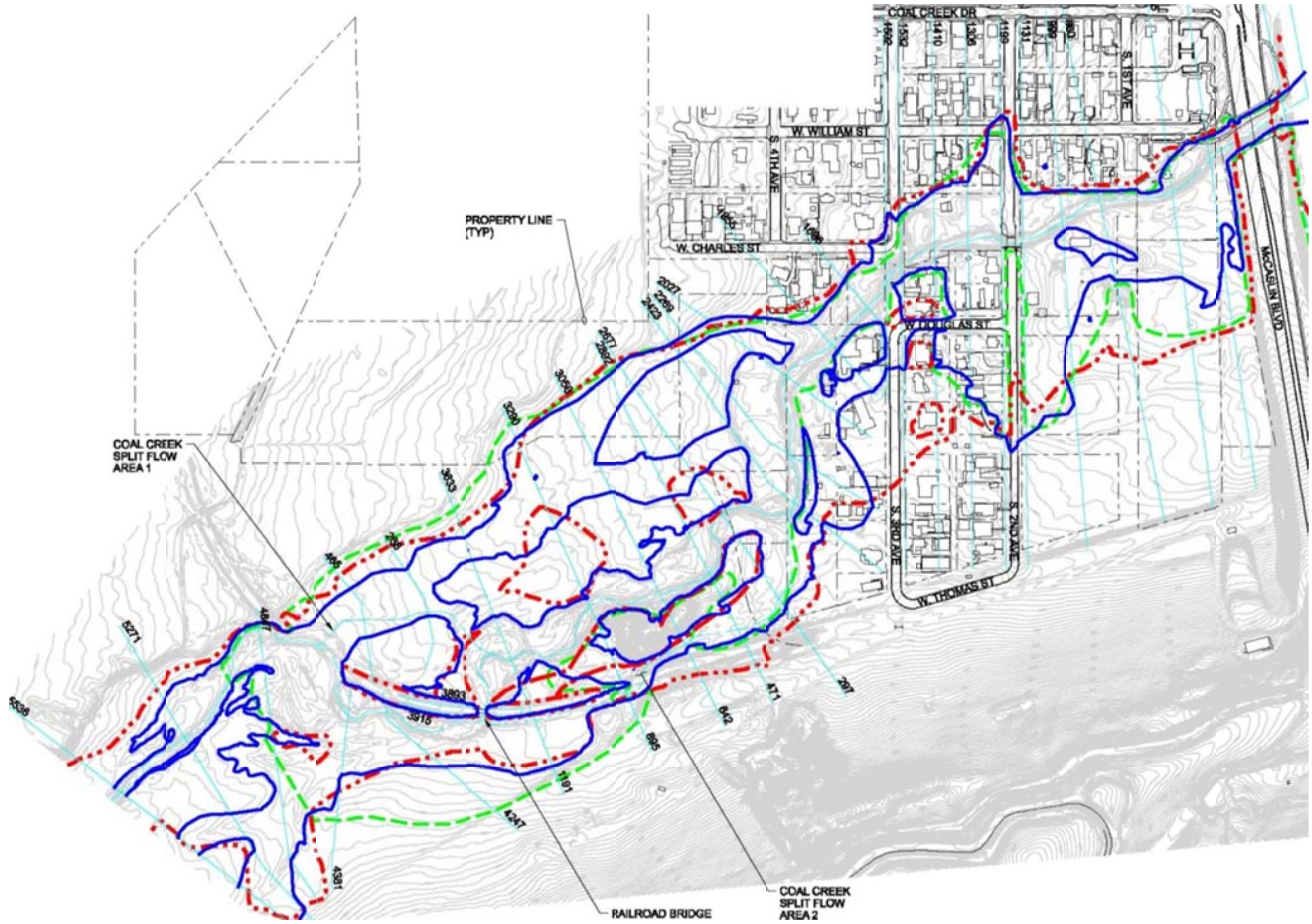

Figure 3-2: Example of the Coal Creek 1-D HEC-RAS FHAD

Table 3-3 contains details of the Coal Creek study area.

TABLE 3-3
Watershed Details for Coal Creek

Drainage Area (acres)	Average Watershed Slope (%)	Typical Development Characteristics	Length of Study Reach (miles)	Peak Flowrate (Q) (cfs)
16,835	1.20	Undeveloped upper watershed with low density residential development in the lower watershed	1	3,770

The 2-D model characteristics for Coal Creek are listed in Table 3-4

TABLE 3-4
2-D Model Parameters for the Coal Creek Watershed

Grid Size (ft)	Number of Grid Cells	Number of Hydraulic Structures	Model Run Time (hours)	Model Development Effort
14	43,000	0	7	Minimal

3.3 Little Dry Creek

A 1-D unsteady flow hydraulic model was developed for the Englewood and Holly Dam Emergency Action Plan (EAP) to determine the flood hazard posed by these two dams along Little Dry creek if they were to experience a sunny day dam failure. Little Dry Creek extends from the outlet of the dams to the South Platte River, passing through City of Centennial, City of Greenwood Village, City of Cherry Hills Village and City of Englewood. This case study was included to determine the effects of grid sizing on model run times and floodplain resolution. Figure 3-3 - Example of the Englewood and Holy Dam EAP 1-D HEC-RAS Analysis, depicts the lower portion of the Dry Creek Stream at the confluence with the South Platte River.

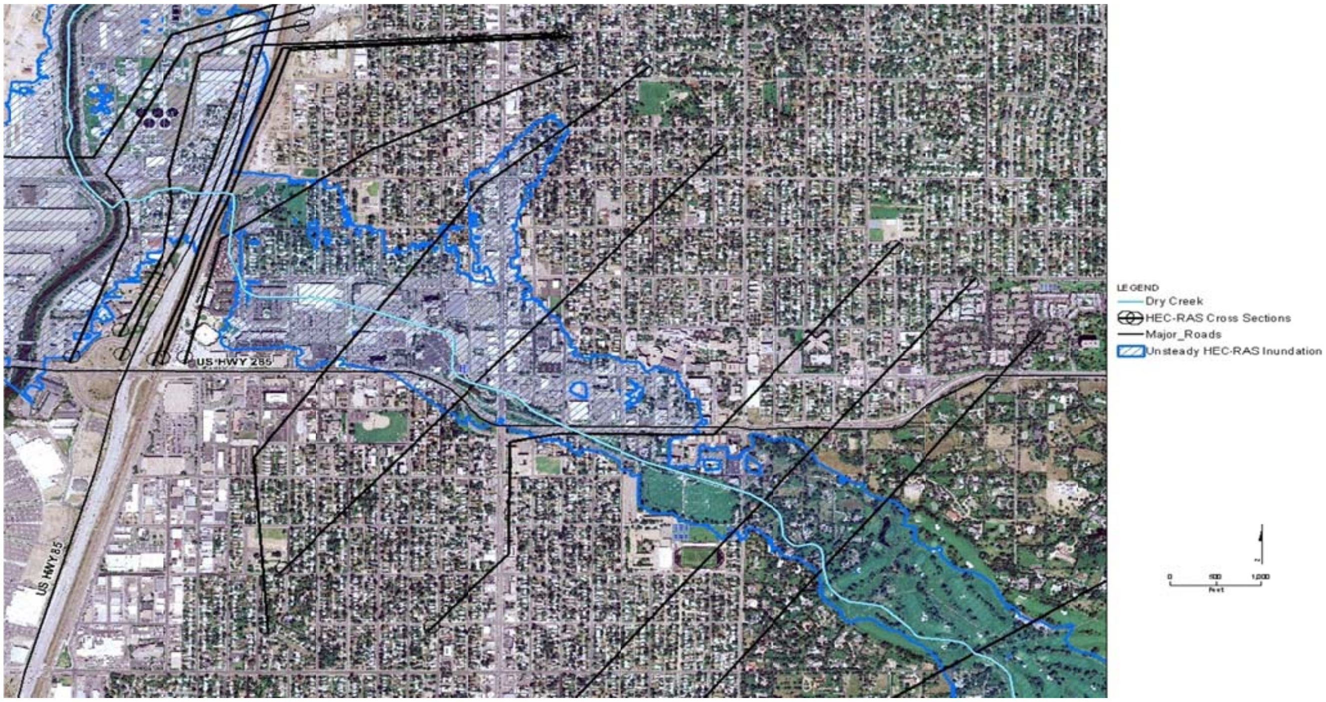

Figure 3-3 - Example of the Englewood and Holy Dam EAP 1-D HEC-RAS Analysis

Table 3-5 contains details of the Little Dry Creek case study.

TABLE 3-5
Watershed Details for Little Dry Creek

Drainage Area (acres)	**Average Watershed Slope (%)**	**Typical Development Characteristics**	**Length of Study Reach (miles)**	**Peak Flowrate (Q) (cfs)**
Dam Breach	0.66	Fully developed watershed with industrial, commercial and residential land use	9	100,000

The 2-D model characteristics for Little Dry Creek are found in Table 3-6.

TABLE 3-6
2-D Model Parameters for Little Dry Creek

Grid Size (ft)	**Number of Grid Cells**	**Number of Hydraulic Structures**	**Model Run Time (hours)**	**Model Development Effort**
50	191,000	0	15	Minimal

4 2-D Modeling Keys to Success

4.1 Grid Cell Sizing

Grid cell sizing has a direct impact on the size of the modeling domain. Large model domains contain a large number (> 60,000) grid cells. Models with large model domains can be expected to run slow with run times up to a day or more. Small modeling domains contain a small number of grid cells (< 15,000) relative to the area being modeled and generally have run times less than 1 hour. The size of the model domain, which is impacted by the size of the individual grid cells, has a dramatic effect on the run time, resolution and relative accuracy of floodplain delineations for 2-D models as well as the general stability of model runs. The effects of grid cell size on 2-D models are discussed below.

4.1.1 Effects on Run Time

To begin the selection of a grid cell size for the development of a 2-D model bathymetric file, model literature recommends that grid cell size be selected so that the estimated peak discharge divided by the area of a single grid cell should fall between (FLO-2D Pocket Guide, 2011):

$$\mathbf{0.1\ cfs/sq\text{-}ft < QPeak/Asurf < 1\ cfs/ft^2} \qquad \textbf{[EQ 4-1]}$$

As an example, for a 10’x10’ grid cell there should be no more than 100 cfs loaded on any particular grid cell. Conversely if your peak flow is 1,000 cfs the recommended grid size is approximately 30’ x 30’. This relationship will help optimize run times and help with the stability of the 2-D model calculations, however applying these recommendations can affect the accuracy of the resulting floodplain by not providing the level of resolution to correctly determine floodplain boundaries and flow splits.

To provide additional guidance on the selection of a grid cell size, three bathymetric grid cell files were created for the Little Dry Creek case study testing the effect of various grid sizes on model run times. Table 4-1 contains the results of the analysis.

TABLE 4-1
The Effect on Run times with Various Grid Cell Sizes

Grid Cell Dimension (ft)	Grid Cell Area (sq-ft)	Number of Grid Cells in Domain	Peak Flow Rate (cfs)	QPeak/Asurf (cfs/sq-ft)	Approximate Model Run Time (min)
300	90,000	5,005	68,000	0.76	2
100	10,000	48,112	68,000	6.8	240
50	2,500	191,257	68,000	27.2	900

The results clearly show that if the grid cell dimension is reduced by 50%, the number of grid cells defining the model domain will increase four-fold. Likewise, the same reduction in grid cell dimension also results in almost a four-fold increase in model run time. Model run times become a factor in implementing 2-D models. Model run time should be considered when determining if utilizing a 2-D model will be efficient for floodplain analysis versus the development of a 1-D model. Based on the analysis from the Little Dry Creek case study, a curve was created to help estimate approximate project run times by utilizing the peak flow rate divided by grid cell area (Q_{Peak}/A_{surf}) criteria. The curve shown in Figure 4-1 indicates that there is almost a linear relationship between Q_{Peak}/A_{surf} and model run time.

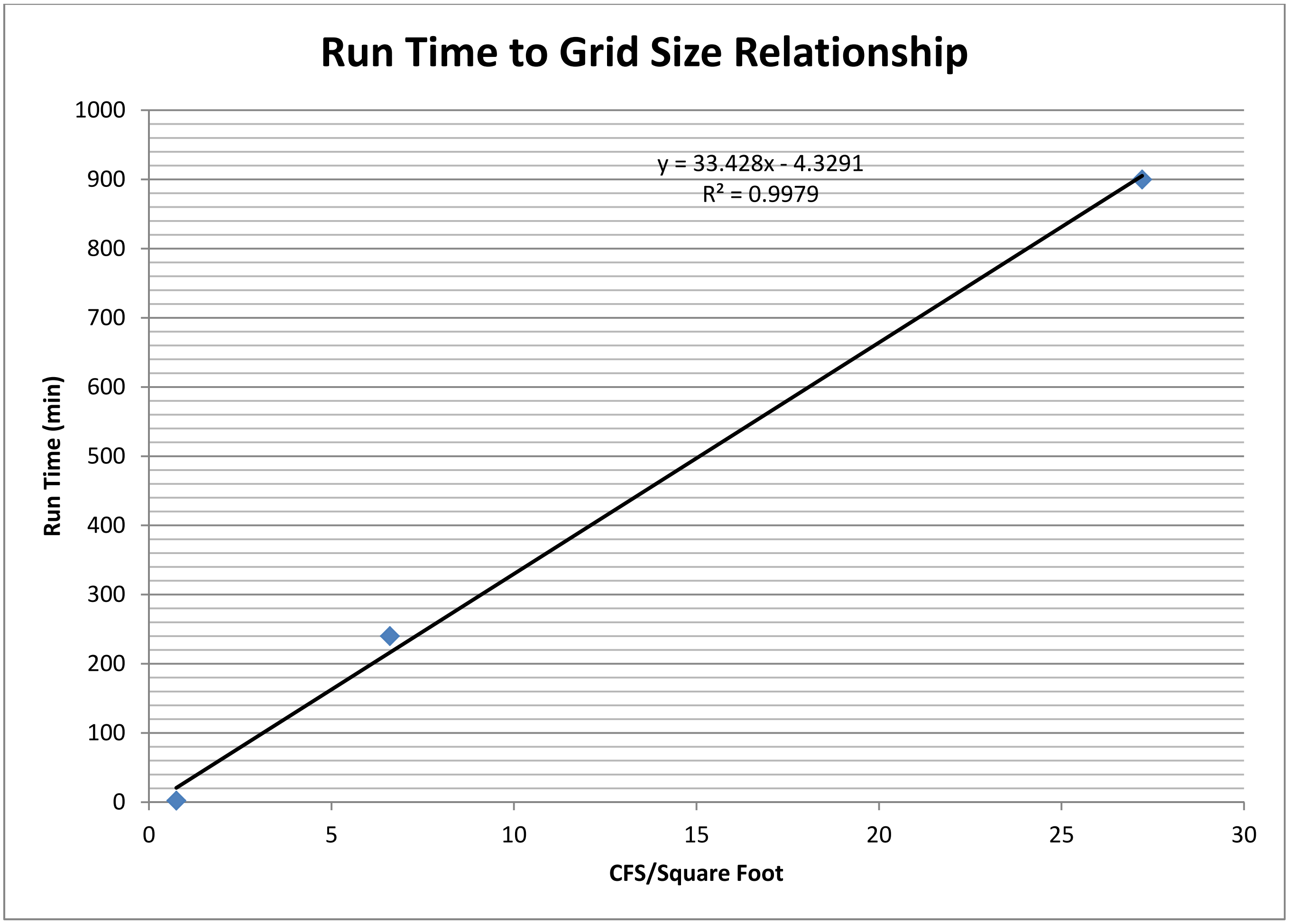

Figure 4-1 - Model Run Time to Grid Size Relationship

4.1.2 Effects on Resolution

Grid cell sizing also has a profound effect on the resolution of floodplain mapping as well as flow path centerline identification. A comparison of floodplain delineations produced using different grid cell sizing was created for the Little Dry Creek case study. Similar to the run time analysis, the resolution of the floodplains was compared between 300-foot grid cells, 100-foot grid cells and 50-foot grid cells. Figures 5-2, 5-3 and 5-4 represent the floodplains that were defined based on the 2-D modeling for the 300-, 100- and 50-foot grid cell 2-D models, respectively.

Grid cell sizing has a profound effect on model set up, run times and resolution. Often these three interests are in conflict. As such, working with project sponsors to help define the requirements of a study prior to developing a 2-D model in order to understand both the potential impacts to project schedules and the quality of the final mapping product is imperative. As a rule of thumb, attempting to size grid cells so that there is a minimum of 2 grid cells per street feature or channel feature tends to be adequate resolution while maintaining reasonable run times. For example, for a 22-foot wide collector street, a grid size of 12 to 16 feet should suffice.

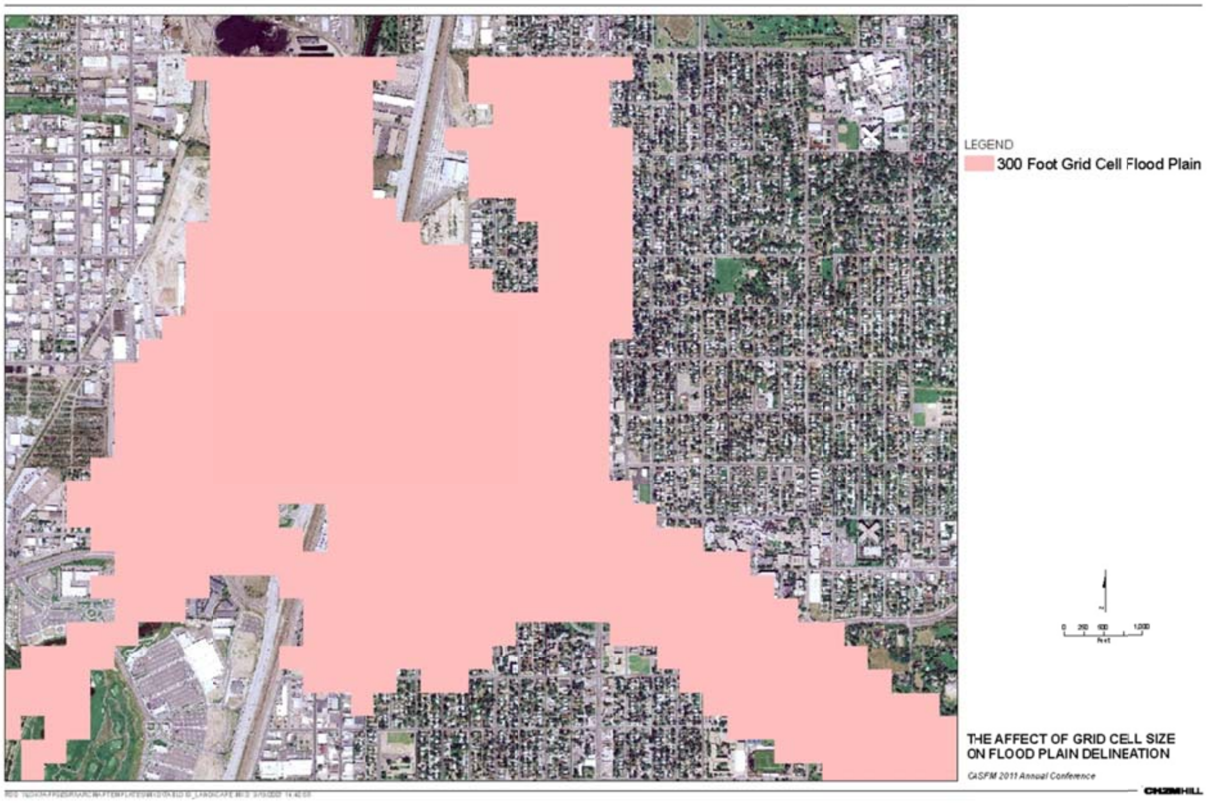

Figure 4-2 - Floodplain Delineation using 300-Foot Grid Cells

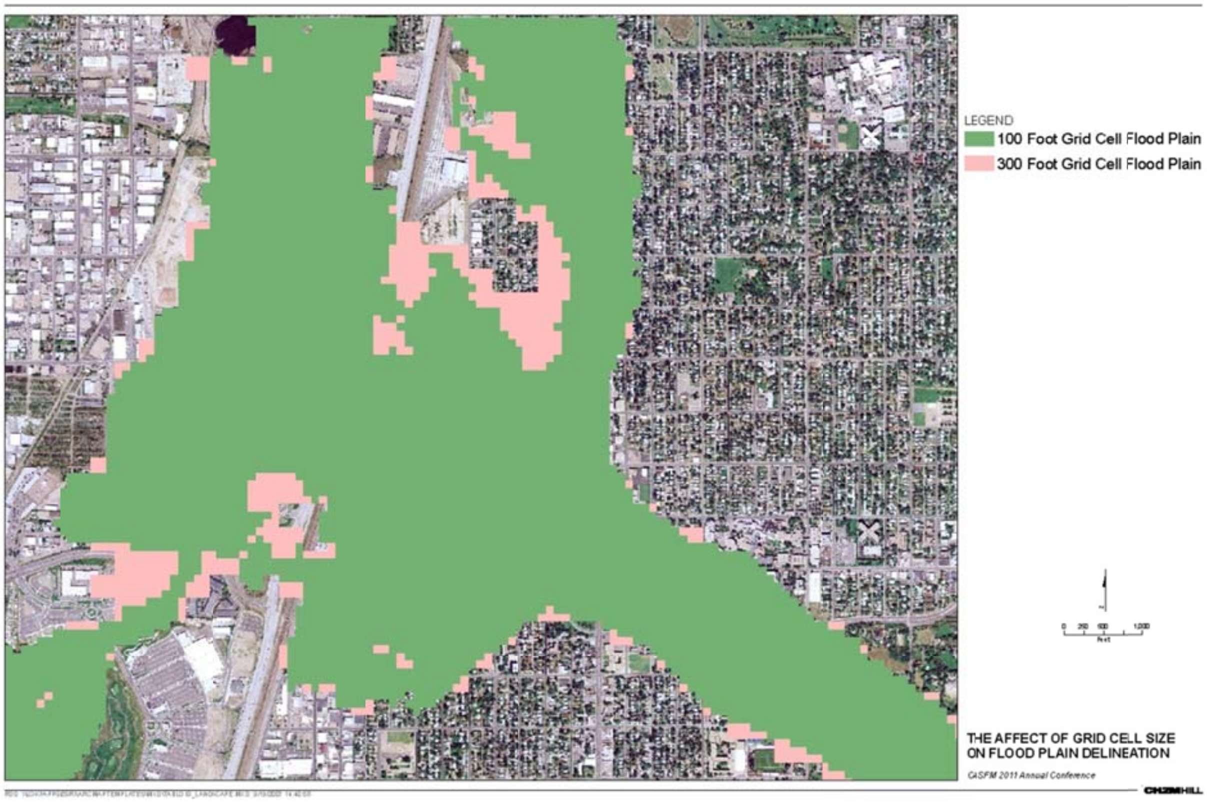

Figure 4-3 - Floodplain Delineation Comparing 300- and 100-Foot Grid Cells

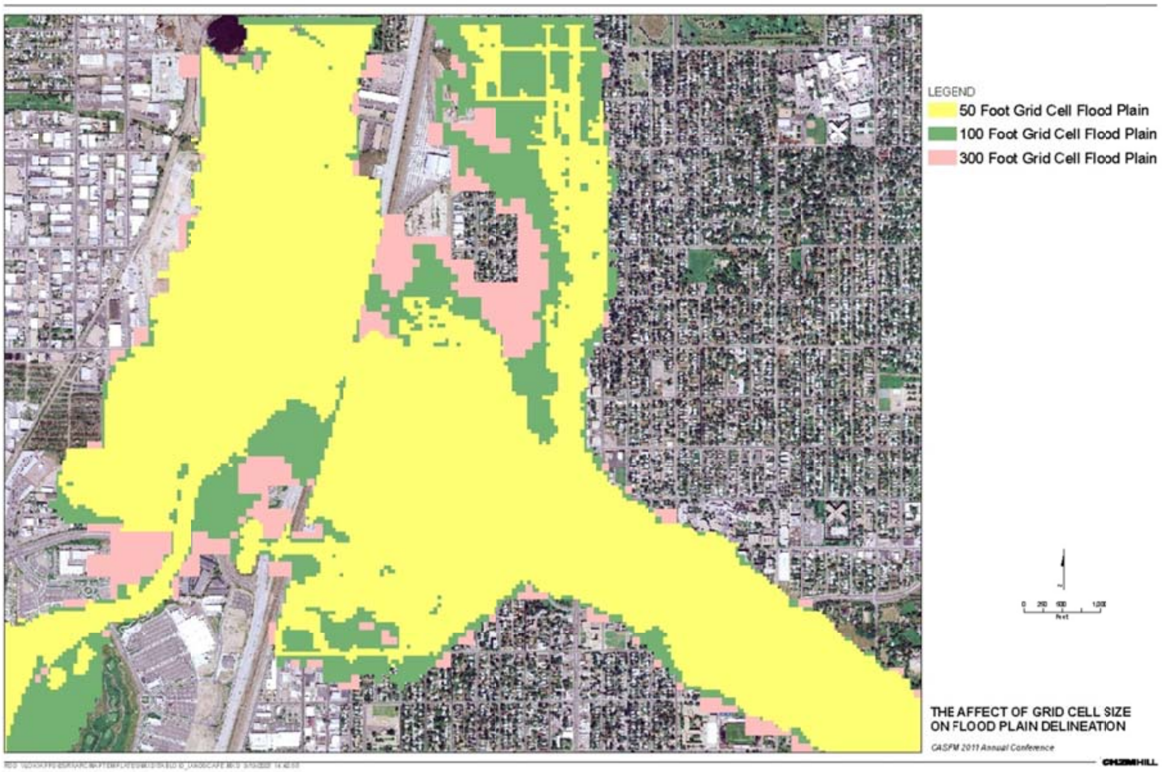

Figure 4-4 - Floodplain Delineation Comparing 300-, 100- and 50-Foot Grid Cells

As can be seen from the figures, there is a significant difference between the resolution and flooding impact between the three models with different grid cell sizes. Large, coarse grid models can be a quick and effective tool for identifying regional tendencies with respect to inundation areas and major split flow paths. However, for the purposes of a detailed floodplain analysis and eventually converting a 2-D model into a 1-D regulatory model, the more detailed results provided by a smaller grid size is needed to help define flow paths. As shown in Figure 4-4, the 50-foot grid cell model for Little Dry Creek better represents flood flow down individual streets than the coarser grid models in Figure 4-2 and Figure 4-3.

4.2 Manning's *n* Value

Manning's *n* value determination is handled differently in 2-D models than in traditional 1-D steady state models. For example, HEC-RAS models define the Manning's *n* value for a minimum of three areas across the cross section representing the left over bank roughness, channel roughness, and right overbank roughness. Although HEC-RAS allows for more than three areas to be defined with a Manning's *n* value, these areas still represent simplifying assumptions regarding the spatial variability of the *n* across the cross section. For 1-D models, streams along the Front Range of Colorado generally have a Manning's *n* value ranging between 0.03 and 0.1 for conditions ranging from bare earth to dense vegetation.

In contrast to a typical HEC-RAS model, for a 2-D model the Manning's *n* is defined independently at every grid cell. These Manning's *n* values are defined generally by land use or land cover for an area and provide much greater spatial variability in the *n* value that a typical HEC-RAS model. In addition, a majority of the flow is calculated as shallow overbank flooding and not dominated by channelized flow characteristics. Therefore, Manning's *n* values can vary greatly depending on the land use conditions of the watershed. For typical riverine ground cover Manning's *n* can vary between 0.17 - 0.80 for dense turf. The U.S. Army Corps of Engineers has provided guidance in the *Introduction and Application of Kinematic Wave routing Techniques Using HEC-1, July 1993*, (TD-10) for Manning's *n* values that are echoed in many 2-D modeling programs. Figure 4-5 contains Table 1 from TD-10.

As can be seen from Figure 4-5, there are a wide range of values for the overbank flooding Manning's *n* values. Engineering judgment will be required to select defensible Manning's *n* values for 2-D models with consideration to the projected depth of flow across the grid. For more channelized riverine flow, typical riverine *n* values may be applicable. However, in areas of shallow flooding, considerations for the impact of trees, vegetation, cars, structures and other obstacles impeding flow should provide the basis for increasing *n* values.

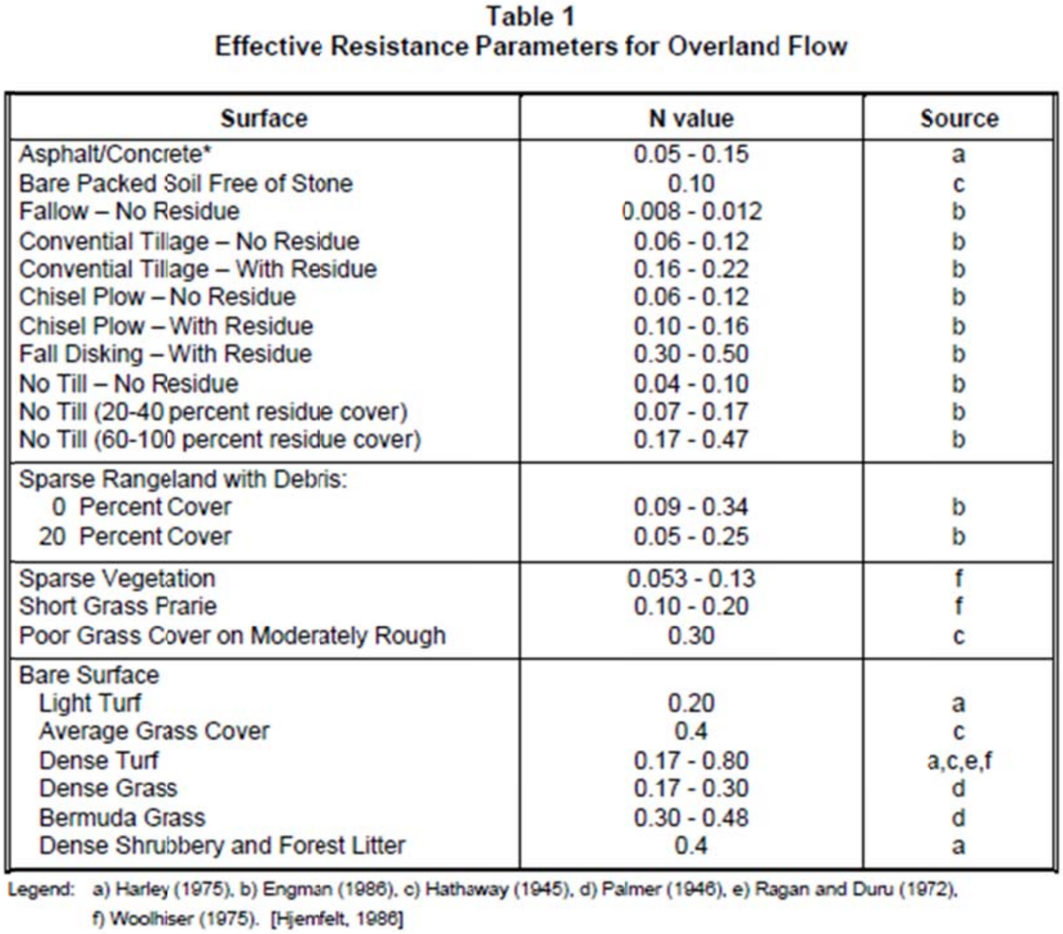

Table 1
Effective Resistance Parameters for Overland Flow

Surface	N value	Source
Asphalt/Concrete*	0.05 - 0.15	a
Bare Packed Soil Free of Stone	0.10	c
Fallow – No Residue	0.008 - 0.012	b
Convential Tillage – No Residue	0.06 - 0.12	b
Convential Tillage – With Residue	0.16 - 0.22	b
Chisel Plow – No Residue	0.06 - 0.12	b
Chisel Plow – With Residue	0.10 - 0.16	b
Fall Disking – With Residue	0.30 - 0.50	b
No Till – No Residue	0.04 - 0.10	b
No Till (20-40 percent residue cover)	0.07 - 0.17	b
No Till (60-100 percent residue cover)	0.17 - 0.47	b
Sparse Rangeland with Debris:		
0 Percent Cover	0.09 - 0.34	b
20 Percent Cover	0.05 - 0.25	b
Sparse Vegetation	0.053 - 0.13	f
Short Grass Prarie	0.10 - 0.20	f
Poor Grass Cover on Moderately Rough	0.30	c
Bare Surface		
Light Turf	0.20	a
Average Grass Cover	0.4	c
Dense Turf	0.17 - 0.80	a,c,e,f
Dense Grass	0.17 - 0.30	d
Bermuda Grass	0.30 - 0.48	d
Dense Shrubbery and Forest Litter	0.4	a

Legend: a) Harley (1975), b) Engman (1986), c) Hathaway (1945), d) Palmer (1946), e) Ragan and Duru (1972), f) Woolhiser (1975). [Hjemfelt, 1986]

Figure 4-5 - Manning's n Value Recommendation from Table 1 of the

Introduction and Application of Kinematic Wave routing Techniques Using HEC-1, USACE 1993

4.3 Modeling Structures

There are three ways in which 2-D models incorporate culverts and other hydraulic control structures into the modeling domain. They are:

1. Incorporating a short 1-D coupled model into the 2-D model so that flow is transferred from the 2-D domain, to the structure that is analyzed as a 1-D hydraulic structure. Flow is then passed back from the 1-D hydraulic structure into the 2-D modeling domain.
2. Inputting rating curves developed from external hydraulic computations that relate head to discharge.
3. Modifying the model’s bathymetric grid to "burn" the culvert into the elevation grid and manipulating Manning's *n* values and area reduction factors to mimic the hydraulic performance of a culvert as determined by external hydraulic computations.

The three methods have different levels of complexity. Incorporating a 1-D coupled model requires the addition of inlet and outlet points as well as the creation of 1-D cross sections to embed into the 2-D modeling domain.

Depending on the number of hydraulic structures, this can be a labor-intensive process and can be the source of instabilities when running the 2-D model.

Figure 4-6 depicts a 2-D model run on the Willow Creek channel system that includes both 1-D results (red lines) and 2-D results (blue and green shading). Rating curves developed from the HY-8 computer program developed by the Federal Highway Administration (FHWA) were used to represent culverts in this area. As shown by the red linework developed by a HEC-RAS model the culverts have the capacity to pass the 100-year flow. However, as shown by the green and blue shading, the 2-D model is depicting undersized culverts and extensive overtopping.

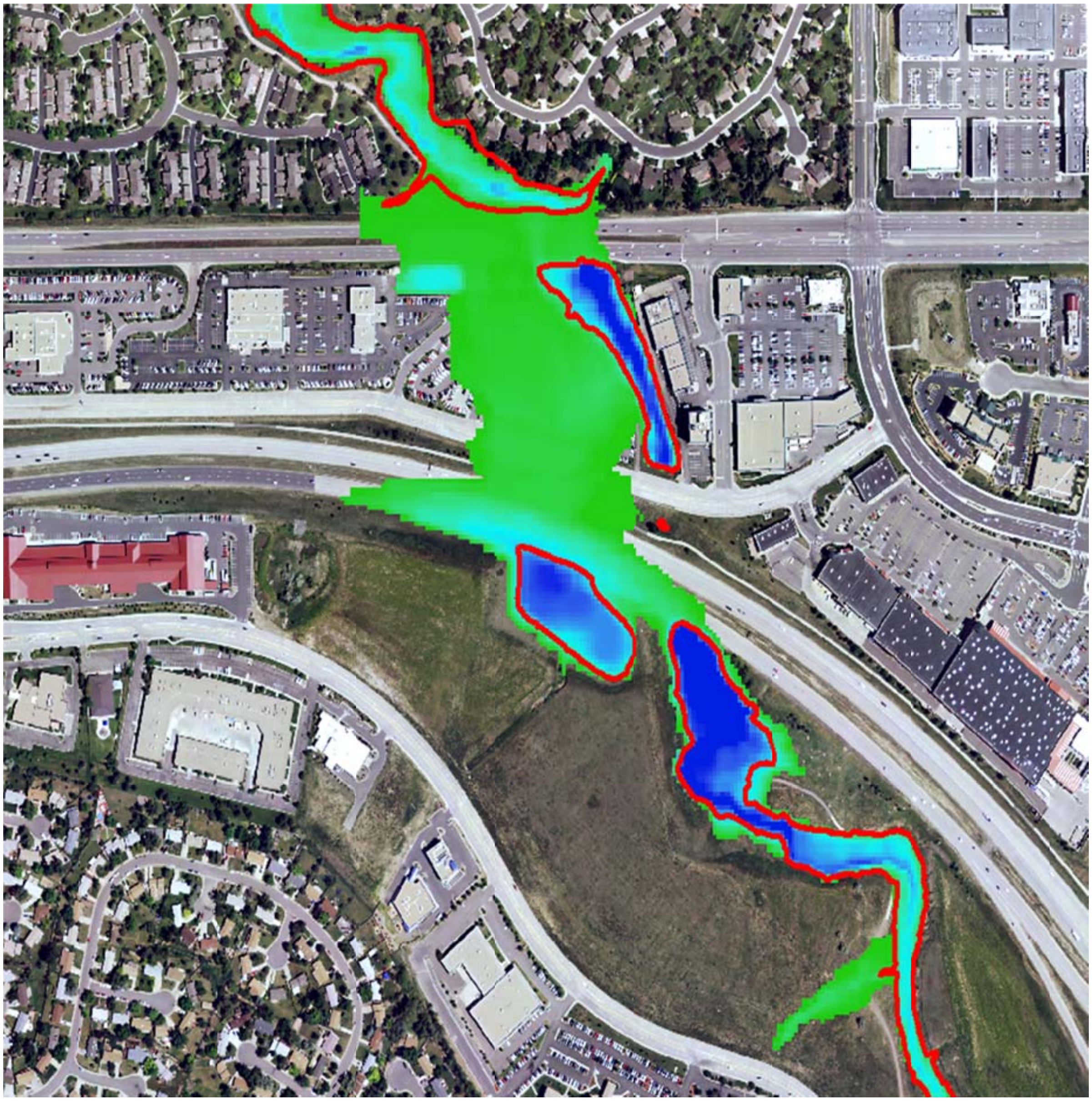

Figure 4-6 - Willow Creek Example with Rating Curves used to Model Hydraulic Structures

"Burning" the culverts into the terrain and changing the Manning's *n* value to replicate the culvert material is a quick and accurate way to represent culverts. Figure 4-7 depicts the same stretch of Willow Creek with the culverts burned into the bathymetric data. Care was taken to adjust area reduction factors, a factor found in the 2-D model to reduce the grid cell area available for conveyance, to replicate the area of the culverts. This 2-D model was closer to replicating the 1-D results shown in red.

For quick analysis, "burning" hydraulic structures can be an efficient way to replicate hydraulic structures in 2-D models and is recommended for the analysis of riverine systems that will ultimately be regulated using a 1-D model. Careful consideration of the culvert barrel cross-sectional area, Manning's *n* values and culvert lengths need to be considered to accurately replicate culverts using this methodology.

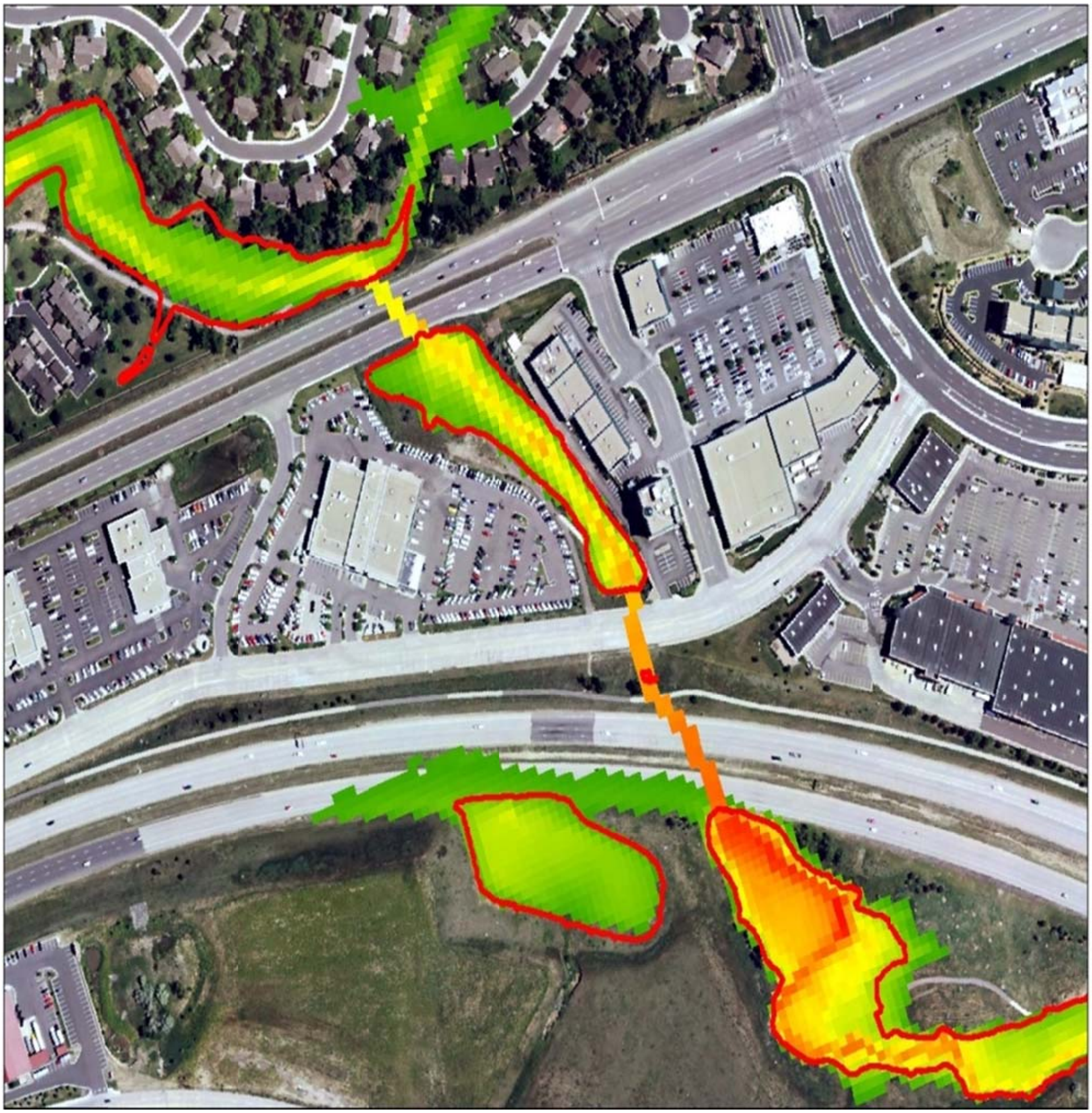

Figure 4-7 - Willow Creek Example with Culverts "Burned" Into Grid

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5 Creating 1-D models from 2-D Results

The ability to quickly develop 2-D models enables professionals to utilize the 2-D model results to develop more accurate 1-D models that can be shared in the engineering and regulatory community. The results of the three case studies indicate that there are four specific applications in which to use the results of a 2-D model to develop a more accurate 1-D riverine hydraulic model.

5.1 Flow Path Centerline Identification

The foundation of any 1-D model is identifying the stream centerline or major flow path, which is used to orient cross sections. For channels with split flows resulting in multiple flow paths or for areas of shallow flooding without a defined flow path, it may be difficult to clearly ascertain the true centerline when initially developing the model. A 2-D model can assist in these situations by displaying the flow direction with flow vectors, as shown in Figure 5-1.

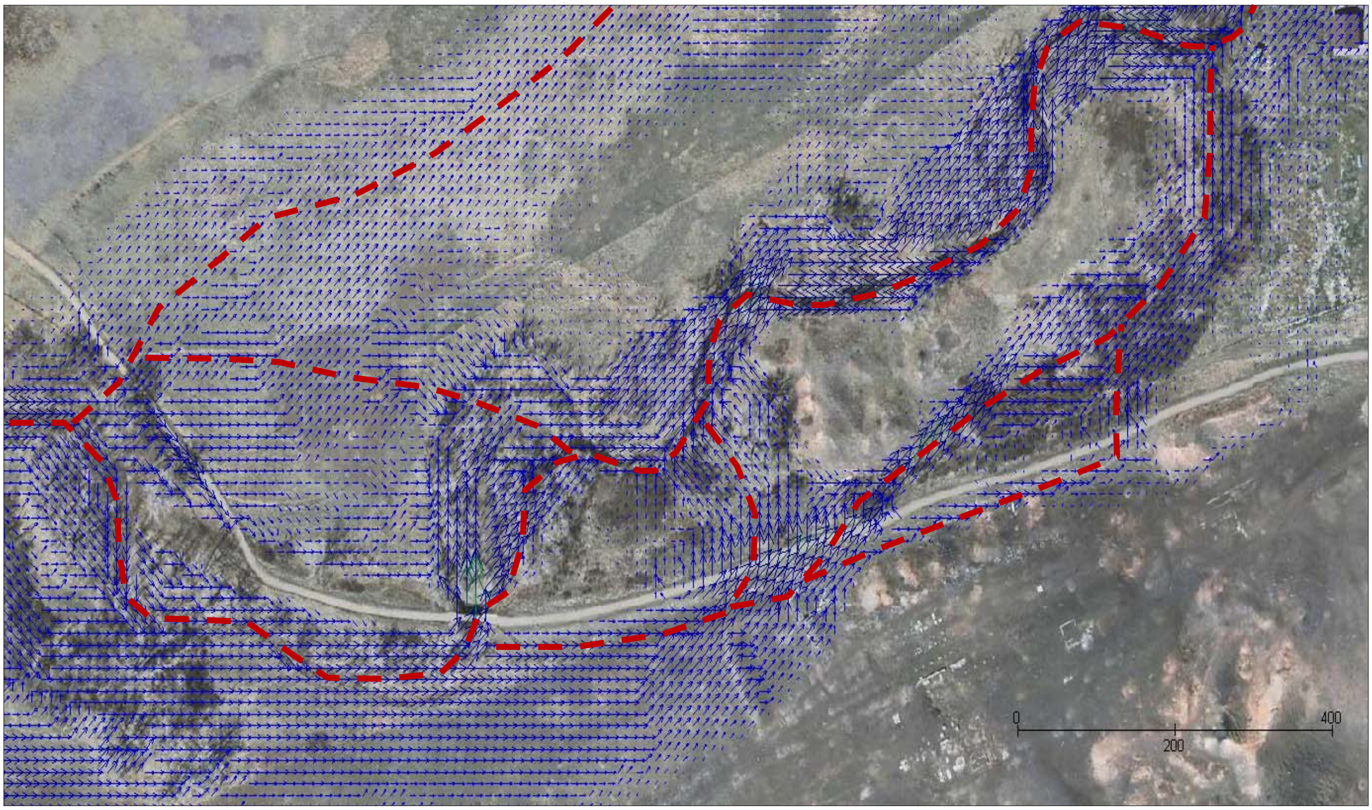


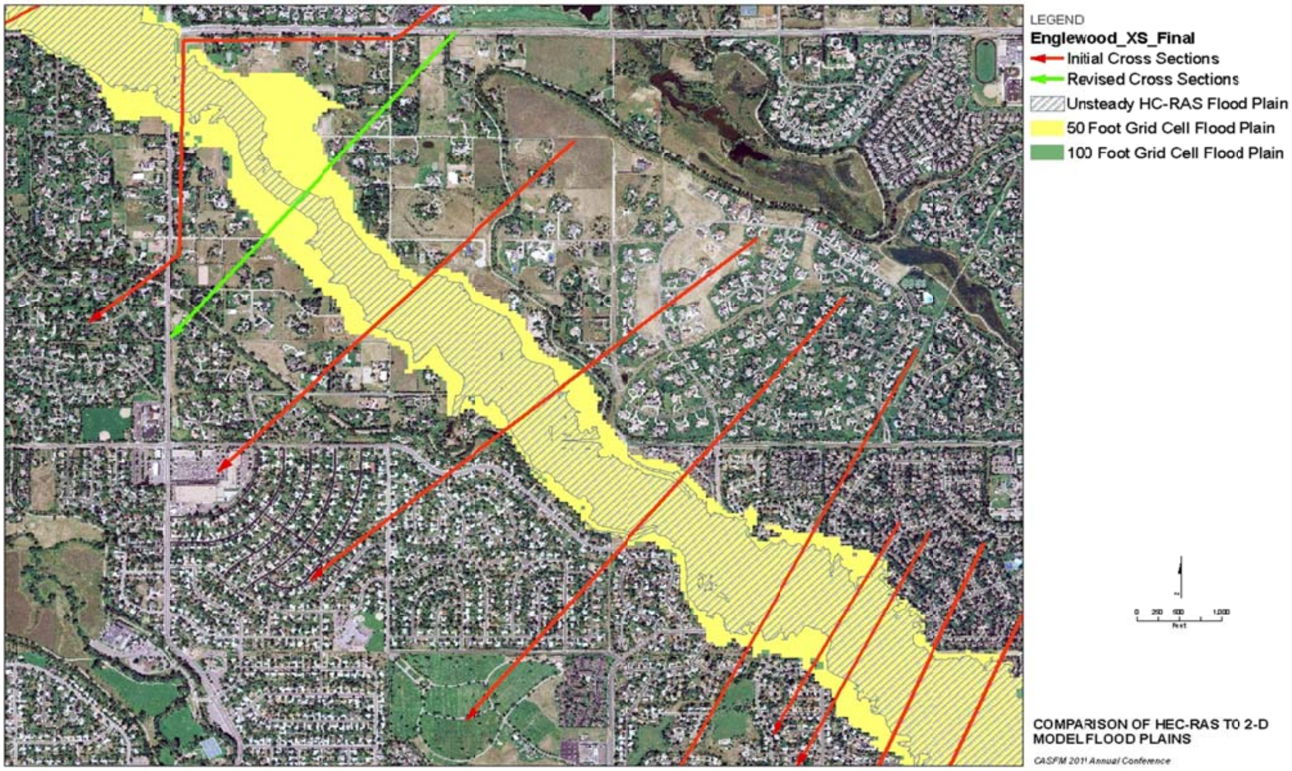
Figure 5-1 - 2-D Flow Vectors Displayed for Coal Creek

The flow vectors indicate the major flow direction so the channel centerline can be easily identified, eliminating the guesswork involved when setting up a 1-D model. The flow vectors also identify where flow splits occur and provide a visual representation of how flows react in a flood condition. Utilizing the flow vectors to identify the flow paths allows the modeler to correctly assign the channel centerlines for use in development of the 1-D model, eliminating an iterative approach that is often necessary in a traditional 1-D modeling approach.

5.2 Cross Section Locations

Once the channel centerlines have been identified in the 1-D model, the cross section layout can then be determined utilizing the same flow vector output from the 2-D model. The cross sections can be drawn perpendicular to the flow path as depicted by the centerline and extended far enough to contain the flow limits, as determined by the flow vectors and 2-D floodplain limits. The flow vectors and floodplain limits also identify variation in flow direction in a floodplain, allowing the modeler to determine where to bend cross sections if necessary.

The flow vector output from the 2-D model also identifies locations where hydraulically significant events occur, such as narrowing of the floodplain, flow acceleration or deceleration, and mounding behind embankments or structures. This enables the modeler to put cross sections in appropriate places to create an accurate 1-D model.



COMPARISON OF HEC-RAS TO 2-D MODEL FLOOD PLAINS

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Figure 5-2 – Little Dry Creek Example of Cross Section Location Selection

The Little Dry creek 1-D model originally placed cross sections shown in red on Figure 5-2. The results of the model showed a significant narrowing of the floodplain where no cross sections were placed. After the results of the 2-D model were viewed, showing a wider flow path a new cross section was added to the 1-D model, shown in green. The revised 1-D model results reflected the wider flow path providing a more accurate model.

5.3 Split Flow Rate Determination

The 2-D model not only identifies direction of flow in split flow situations but also determines the amount of flow going in each direction. As discussed previously, the 2-D model is a dynamic model that routes a volume of water through the system, as opposed to a static 1-D flood model that computes flow depth for a constant flow rate at each cross section. When flow splits occur, the 1-D model requires weir equations, orifice equations, a combination of the two equations, or balancing head to determine the amount of flow at a split. This is much different from a 2-D approach, which continues to utilize the Saint-Venant equations to compute flow in any direction.

The Coal Creek case study had multiple split flows from the main channel that required multiple iterations for the 1-D model development. The 2-D model for the Coal Creek study allowed the model to identify the quantity and direction of flow splits. The 2-D model identified eight (8) separate flow splits that occur. These eight split flows were evaluated for both volume and peak flow to determine if a relationship existed that could be used to determine appropriate steady state peak flows to utilize in 1-D models with split flows.

In this analysis, the Split Volume (Vsplit) was compared to the Total Volume (Vtotal), and the Flow Rate Split (Qsplit) was compared to the Total Flow (Qtotal). The goal was to identify a relationship between the dynamically routed volumes or flow split that could then be used to compute the flow split in the steady state 1-D model. The results of the split flow analysis are shown in Table 5-1.

TABLE 5-1
Flow Ratio and Volume Ratio for split flows Identified on Coal Creek

	Split 1	Split 2	Split 3	Split 4	Split 5	Split 6	Split 7	Split 8
Q_{Split}/Q_{Total}	20%	53%	47%	44%	56%	44%	49%	7%
V_{Split}/V_{Total}	17%	53%	47%	38%	62%	41%	51%	6%

The results of the analysis show there is similarity in the ratios of both split volume and split flow rate when compared to the whole. The most significant differences occur where water ponds behind structures causing volumes of water to be stored thus reducing the volume split. Due to this impact, inadvertent detention behind embankments has on volumes, it is recommended to utilize the split flow rate percentage to compute the recommended flow rates for split flows in the 1-D model. This allows the modeler to hard code into the 1-D model the flow rates in each reach, rather than utilizing the limited capabilities of the 1-D model to determine the weight of flow splits. Figure 5-3 shows the difference between the flow rates determined utilizing the traditional 1-D modeling approach (blue arrows) and the more accurate results from a 2-D model (yellow arrows).

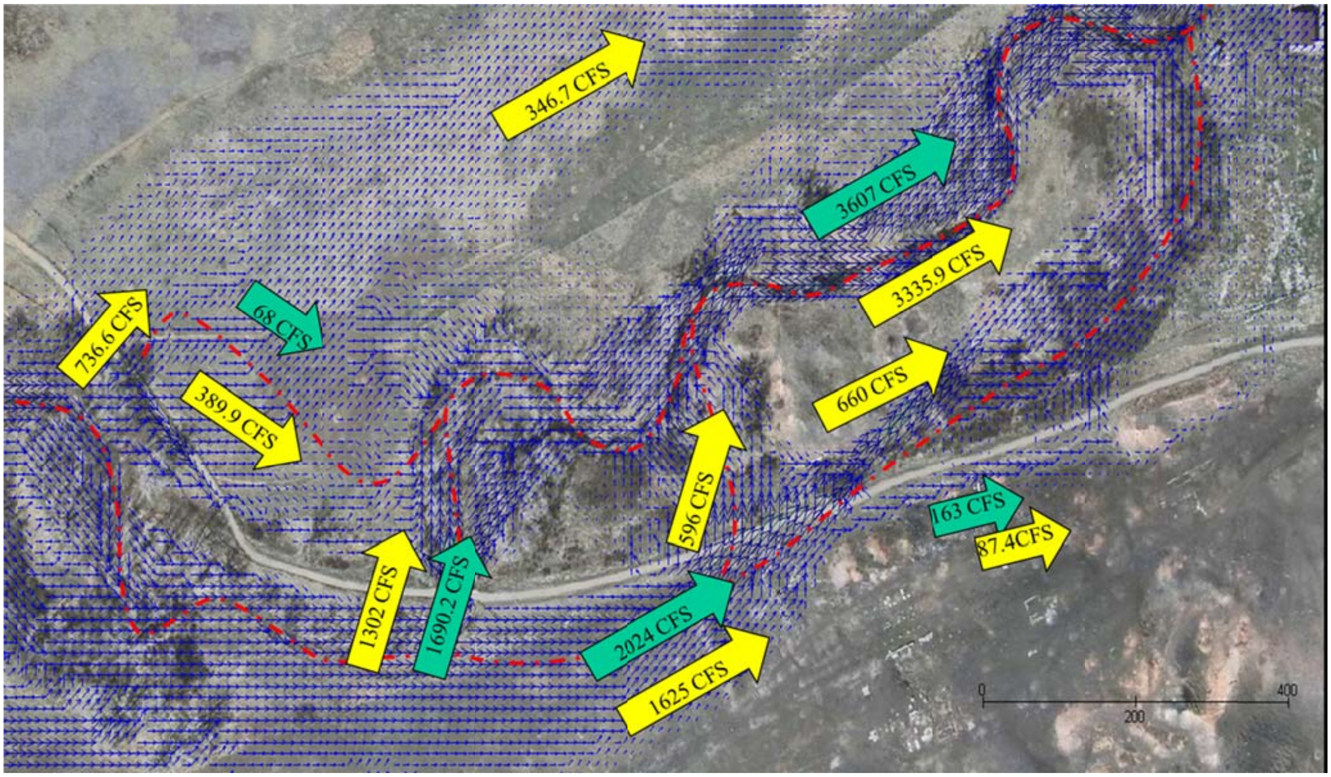

Figure 5-3 - Differences between Flow Rates from Traditional 1-D model and 2-D Model for Coal Creek

The results of the Coal Creek analysis determined that the flow splits from the 1-D model are different from the 2-D results in both direction and quantity. The complex split flow characteristics of Coal Creek show the difficulty in accurately computing flow splits with a steady state 1-D model. The applications of the results from this analysis are discussed in Section 6, Conclusions and Recommendations.

5.4 Floodplain Delineation

The way in which 1-D models and 2-D models delineate floodplains are differently. Because the 1-D model computes water surface elevations at each cross section based on the peak flow rate, the floodplain is delineated by intersecting the water surface Triangulated Irregular Network (TIN) with the ground surface TIN to produce a floodplain polygon. These boundaries need to be reviewed for consistency with the HEC-RAS cross section water surface elevation and top width reported in the output and corrected as necessary, but generally, a majority of this work can be completed with automated processes. In contrast, 2-D models compute a water surface elevation at every individual grid cell. When delineating the maximum floodplain extents, 2-D models report the maximum water surface elevation computed at any given time step for any given grid cell. If a grid cell gets wet during only one time step, that cell will be included in the floodplain. Therefore, engineering judgment needs to be exercised when delineating floodplains from a 2-D model.

In general, the delineation of floodplains from a 2-D model should include determining the floodplain boundary for depths less than 0.25 feet (3 inches). Flow depths less than 3 inches can generally be considered nuisance flows. However, the floodplain needs to be reviewed to ensure continuity is maintained for flooding areas; for example, ensuring islands and pockets of deeper flooding are connected to the floodplain.

6 Conclusions and Recommendations

The current regulatory environment in the U.S. for floodplain modeling and development predominately revolves around 1-D modeling tools. Even though 2-D models are accepted for floodplain models, legacy model tie in issues and regulations governing floodplain development are geared toward the use of 1-D models. In order to better define the use of 2-D models this paper focused on three major goals.

1. Review when and how 2-D models should be used;
2. Provide recommendations on 2-D model development;
3. and provide a set of recommendations for the construction of 1-D regulatory models from 2-D models.

6.1 When to use 2-D models

Based on the analysis performed for this paper, it is clear that for riverine conditions in which flows are confined within the channel banks and do not exhibit wide floodplains or split flow conditions, 1-D models provide a better representation of the flow characteristics and generally take less effort to develop than a comparable 2-D model. This is particularly true in channels that have hydraulic structures, since the 1-D modeling routines for structure crossings provide a better representation of the structure hydraulics. The representation of the cross-sectional geometry of the channel is also better characterized by the surveyed cross sections used for a 1-D model than those produced by the bathymetric grid for a 2-D model due to the loss of resolution that can occur as shown in Figure 6-1.

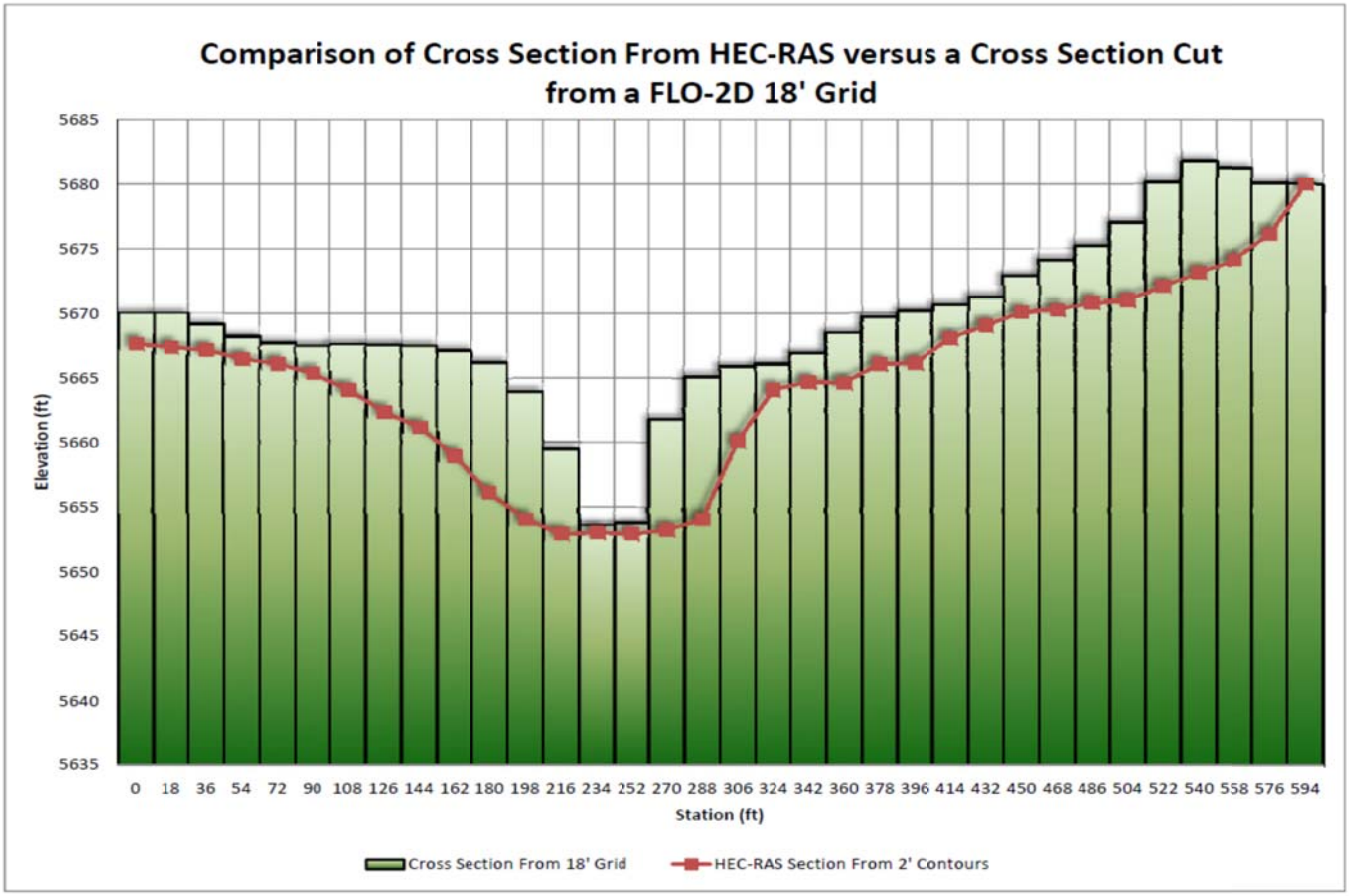

Figure 6-1 - Differences between Cross-Sections from Traditional 1-D model and 2-D Model

However, the benefits associated with performing 2-D modeling on riverine systems can be significant for areas that are known to or are suspected to have multiple flow paths and difficult channel and overbank hydraulics. These benefits include reduced modeling effort to determine the correct split flow locations and flow rates as well as preliminary floodplain information and depths.

In summary, the following guidance is recommended.

1. For riverine systems that are channelized and in general do not have multiple flow paths, 1-D models should be used. 1-D models should be used in these cases because the model development and run time is less cumbersome and the models provide results that are consistent with the current regulatory requirements.
2. For those systems with the data available to create a 2-D model for complex flooding situations a 2-D model can be quickly built and provide additional information for creation of a refined 1-D model.

6.2 Summary 2-D modeling Guidelines and Recommendations

From the analysis completed for this report, it was determined that there are several keys to success that will help in quickly and efficiently develop a 2-D model.

1. Collect the data required for a 2-D model. This includes:
    - Land Use data for the development of Manning's *n* values
    - Topographic data for development of the 2-D elevation file
    - Hydrology and Hydrographs
2. Select the grid cell size for your model. Care needs to be taken to select a grid cell size that is small enough to provide the required floodplain detail, but large enough to ensure a reasonable run time. As a rule of thumb for 100 cfs, the grid size should be approximately 10' x 10'. In addition, every 10-fold increase in flow there should be a corresponding increase in grid cell size of approximately 33%.
3. Finally, careful consideration of hydraulic structures needs to be made. Hydraulic structures can be modeled in three ways.
    - Incorporating a short 1-D coupled model into the 2-D.
    - Inputting rating curves developed from external hydraulic computations that relate head to discharge.
    - Modifying the model’s bathymetric grid to "burn" the culvert into the elevation grid and manipulating Manning's n values and area reduction factors to mimic the hydraulic performance of a culvert as determined by external hydraulic computations.

   The recommended methodology for quick modeling of hydraulic structures is to "burn" the structures into the bathymetric data. This provides and adequate representation of the hydraulic structures and maintains hydrologic continuity and will provide an adequate floodplain analysis to develop and build a 1-D model from.

6.3 Summary of Techniques for creating a 1-D Model from a 2-D model

By utilizing the flow vector results and the flow split percentages a highly accurate 1-D model can be created that can be regulated, shared, and used for floodplain management. The procedure to create a 1-D model from the 2-D model includes the following steps.

1. Identify major flow paths from the 2-D model to delineate stream centerlines for the 1-D model utilizing flow vectors from the 2-D model.
2. Utilizing the flow vectors and project contour data, identify and align cross sections for the 1-D model.

3. Using the flow information, determine the peak flow rate for each stream centerline. To do this take the ratio of Q_{Split}/Q_{Total} developed from the 2-D model.

The results of applying this approach are shown in Figure 6-2 for the Coal Creek watershed. The revised 1-D floodplain utilizing the split flows computed through the 2-D analysis is shown in the purple line, while the 2-D depth raster is shown in green, yellow and red. It should be noted that the 2-D model showed a very shallow overland flow path at the downstream end of the project reach. In the 1-D model development it was determined that the shallow nature of the flow (less than 3”) did not best represent the surface conditions, therefore was not included in the floodplain.

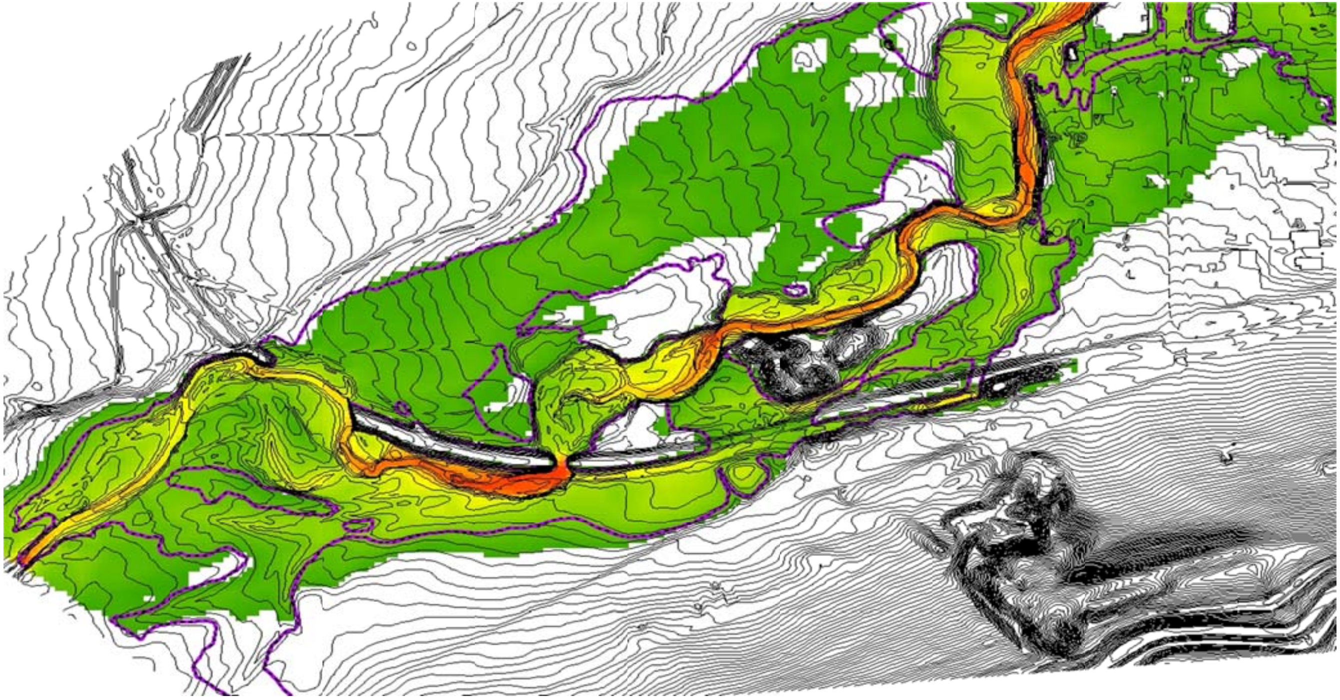

Figure 6-2 – Revised 1-D Floodplain Results Compared to 2-D Model Results

For riverine hydraulic analysis, a 2-D model can be effectively utilized to determine flow paths and critical hydraulic areas. The results of the model can also be used to build a more refined 1-D model through the use of split flow quantity determination, flow path selection and cross section locations. Riverine systems vary greatly by watershed and ultimately the engineer should utilize the most effective technology to accurately model the hydraulics of the system. The information presented here in can be implemented in a time and cost effective way to provide a more complete understanding of riverine hydraulics.

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