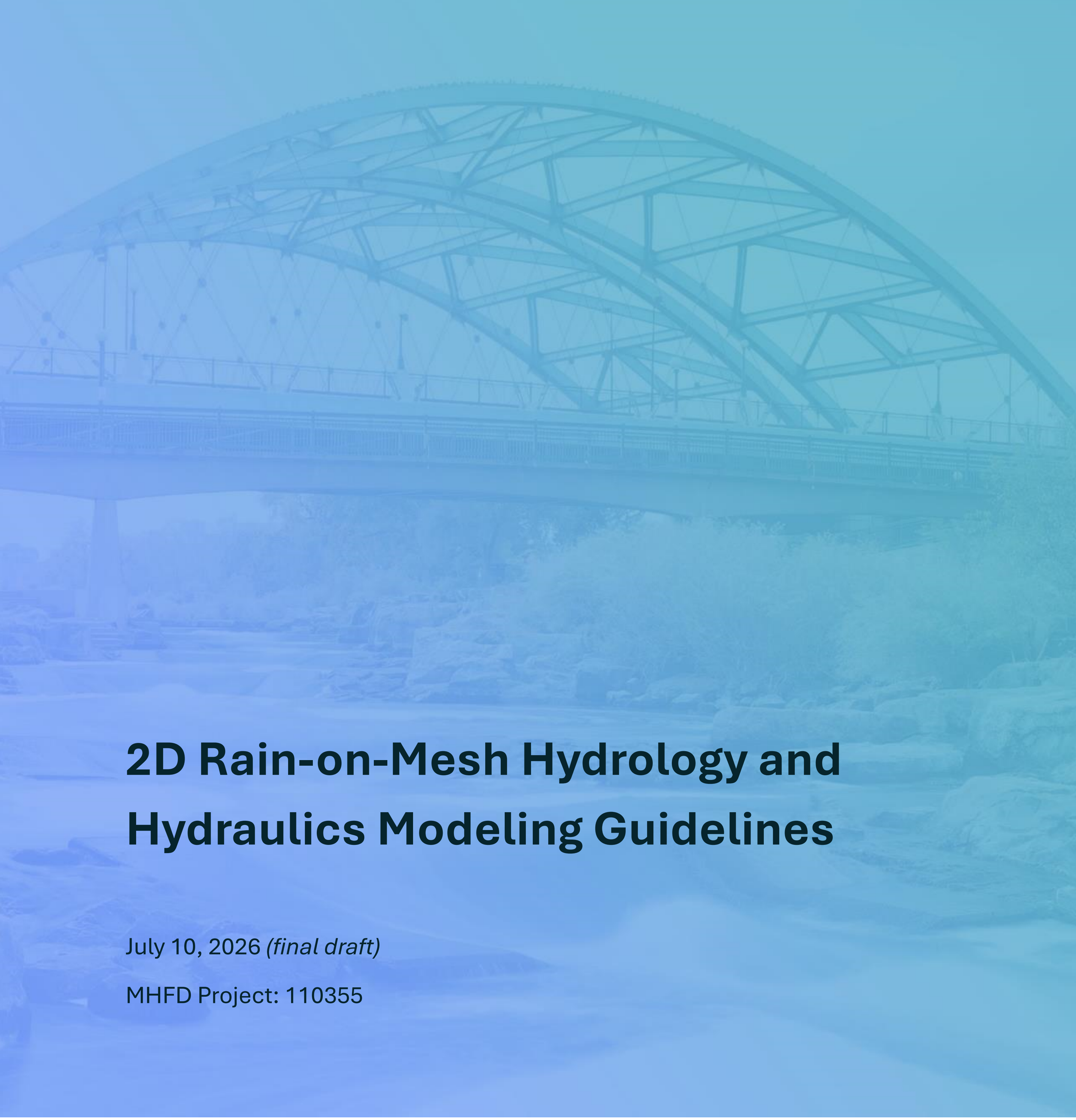

2D Rain-on-Mesh Hydrology and Hydraulics Modeling Guidelines

July 10, 2026 *(final draft)*

MHFD Project: 110355

MHFD
MILE HIGH FLOOD DISTRICT

Michael Baker
INTERNATIONAL

EXECUTIVE SUMMARY

This guidance document defines the Mile High Flood District's (MHFD) recommended approach for two-dimensional (2D) Rain-on-Mesh (RoM) hydrologic and hydraulic (H&H) modeling in support of the MHFD Flood Risk program. Building on prior pilot studies, it establishes consistent expectations for model development, calibration, documentation, quality control, and deliverables so that results are defensible, repeatable, and comparable across watersheds and over time.

- **Intended users:** consulting engineers, contractors, and MHFD technical staff developing, reviewing, or managing 2D RoM studies.
- **Primary objectives:** standardize inputs and workflows; reduce review-cycle ambiguity; improve efficiency; and support consistent flood hazard and risk products for portfolio planning and communication.
- **Relationship to Federal Emergency Management Agency (FEMA):** FEMA Guidance and Standards provide a flexible national framework; this document adds MHFD-specific requirements, prescriptive practices, and reviewer expectations where needed.

Key Requirements and Decision Points

- **Software baseline:** 2D RoM models shall be developed and maintained in **HEC-RAS 7.0 or later**, including updates to legacy models, to support required capabilities such as pipe network representation.
- **Design storms:** run synthetic events for **13 recurrence intervals** (1-, 2-, 5-, 10-, 25-, 50-, 100-, 200-, 300-, 400-, 500-, 750-, and 1000-year), using NOAA Atlas 14 precipitation with the Colorado Urban Hydrograph Procedure (CUHP) with interpolation for non-Atlas intervals as needed and where durations vary depending on drainage area and recurrence interval.
- **Conservative initial conditions:** RoM design storms must use a **restart file** (typically generated from a ≥50-year event run and drained over ~7 days) to consume micro-storage consistent with District policy.
- **Domain sizing and run-time targets:** delineate domains to balance stability and efficiency (urban domains generally ≥7 sq. mi.; larger rural domains may exceed 100 sq. mi.) and target **<24-hour** runtimes for design storms.
- **Typical mesh guidance:** nominal mesh of ~**50 feet** in urban areas (and **100–200 feet** in lower-development areas) with refinements (generally **25–50 feet**) along channels, floodplains, structures, and key breaklines.
- **Stormwater systems:** represent underground stormwater networks using HEC-RAS pipe features; include inlets and pipes for all systems that are geospatially represented.
- **Stability expectations:** keep total volume accounting error **≤2%**; document and justify any main-channel cell water surface elevation error **>0.2 foot**.
- **Mapping and post-processing:** export maximum rasters using sloping, depth-weighted faces at a **0.1 foot** threshold; remove/delete disconnected areas (islands) and fill/close holes less

than 2,500 sq. ft. ; filter nuisance pluvial flooding by intersecting streamlines greater than or equal to 130 acres (0.20 sq. mi.) with rasters to classify intersected continuous flooding as fluvial, and all non-intersected areas as pluvial; and classify fluvial vs pluvial using a **130-acre (0.20 sq. mi.)** drainage threshold.

How to Use this Document

The guidance is organized to follow the typical 2D RoM project lifecycle—from model setup through calibration, production runs, and deliverables. Section 2 provides a summary table of requirements and preferences; subsequent chapters provide the detailed “how” and reviewer expectations.

- **Model setup and documentation:** standardized folder structure, naming conventions, and required metadata to support long-term maintenance and efficient review.
- **Hydrology:** design storm development, calibration/validation precipitation sources (GARR, MRMS, AORC), reservoir and regulation representation, and guidance for basin linking and confluence joint probability analysis where appropriate.
- **Terrain and inputs:** expectations for terrain resolution/quality, bathymetry integration, building representation, and development of land cover, soils, and infiltration inputs.
- **Mesh and geometry construction:** nominal mesh sizing, refinement strategies, breakline enforcement, and representation of structures and stormwater infrastructure.
- **Boundary conditions and computational settings:** internal/external boundary condition practices, recommended solution equations, adaptive timestep control, and stability diagnostics.
- **Calibration, outputs, and QA:** preferred calibration hierarchy and metrics, required design-storm production runs, raster export naming/thresholds, post-processing logic, and deliverable/QC expectations.

Project teams should begin with the summary table in Section 2 to confirm required items and District preferences, then use the detailed chapters to implement and document decisions. Where watershed-specific objectives warrant deviations from this guidance, the alternative approach should be clearly justified, documented, and reviewed/approved by MHFD prior to implementation.

Acknowledgements

This report has been prepared under funding provided by the Mile High Flood District Flood Risk Assessment Program. Its initial draft was distributed on March 23, 2026 and this revised version has been distributed on July 6, 2026 upon incorporated comments from the sponsors and reviewers noted below. A running list of revisions will be maintained as the program evolves and guidance is updated.

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Revision History

This sectional shall be updated as the program evolves and the document is updated.

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List of Acronyms

1D	one dimensional
2D	two dimensional
AE	Zone AE (FEMA flood zone designation)
AEP	annual exceedance probability
AORC	Analysis of Record for Calibration
BC	boundary condition
BLE	Base Level Engineering
CN	Curve Number
CUHP	Colorado Urban Hydrograph Procedure
DAR	drainage area ratio
DEM	digital elevation model
DRCOG	Denver Regional Council of Governments
DSS	Data Storage System
DWE	Diffusion Wave Equation
FEMA	Federal Emergency Management Agency
FFRD	Future of Flood Risk Data
FHAD	Flood Hazard Area Delineation
G&A	Green and Ampt
G&S	guidance and standards
GARR	Gage Adjusted Radar Rainfall
H&H	hydrologic and hydraulic
HEC-HMS	Hydrologic Engineering Center – Hydrologic Modeling System
HGL	hydraulic grade line
JPA	joint probability analysis
LiDAR	Light Detection and Ranging
LPIII	Log Pearson type III
MHFD	Mile High Flood District
MRMS	Multi-Radar/Multi-Sensor
NCHRP	National Cooperative Highway Research Program
NLCD	National Land Cover Database
NOAA	National Oceanic and Atmospheric Administration
NSE	Nash-Sutcliffe Efficiency
QC	quality control
RoM	Rain-on-Mesh
RRE	Regional Regression Equations
SA	storage area
Sq. Mi.	square miles
SSURGO	Soil Survey Geographic Database
SWE	Shallow Water Equations
SWMM	Storm Water Management Model
USDCM	Urban Storm Drainage Criteria Manual
USDA	U.S. Department of Agriculture
USGS	United States Geological Survey

WSE	water surface elevation

1 Introduction and Purpose

This guidance document is intended for use within the Mile High Flood District (MHFD or the District). Its purpose is to outline the recommended approach for two-dimensional (2D) Rain-On-Mesh (RoM) hydrologic and hydraulic (H&H) modeling studies conducted in support of the District's Flood Risk Program. The guidance is informed by prior pilot studies and reflects District preferences, policies, and lessons learned from recent applications.

1.1 Background

MHFD's historical approach to hydrology has been to derive excess runoff from rainfall via the Colorado Urban Hydrograph Procedure (CHUP) and estimate peak flows when routed through the Storm Water Management Model (SWMM). The historical approach to hydraulics has been to apply peak flows to one-dimensional (1D) HEC-RAS models. Technology advances and an evolving regulatory framework has opened the door to two-dimensional (2D) rain-on-mesh (RoM) modeling.

2D RoM modeling provides insights on pluvial flooding. It reduces the number of assumptions and judgements a modeler needs to make by representing the hydraulics, terrain, and watershed characteristics in higher detail, and readily produces gridded outputs for the desired variables to assess risk. This enables a transition from a binary in or out of the floodplain to a more graduated depiction of risk.

To understand the capabilities of RoM modeling in HEC-RAS, in early 2023 the District launched a pilot project for RoM modeling in Niver Creek. The Niver Creek Pilot entailed a comprehensive sensitivity study to establish recommended applied approaches for RoM models, assess their potential regulatory use, and evaluate resulting gridded data to inform project prioritization and risk reduction. The study provided clear guidance for RoM modeling regarding mesh construction, resolution and sources for foundational layers, bathymetry, calibration approaches, model equations and tolerance settings, and approaches for hydraulic structures.

Additional aspects were further explored in a subsequent RoM pilot in Goldsmith Gulch between mid-2024 through late 2025. Goldsmith Gulch further refined the RoM methodology, particularly for infiltration and underground stormwater infrastructure, and validated some of the fundamental differences in results between the prior FHAD approach and new RoM modeling.

Following these two pilots, the initial version of this document was produced in March 2026 based on prior learnings. At that same time, two additional studies were initiated to further refine the methodology at a much larger scale, including Ralston Creek at 90 square miles (sq. mi.) and Second and Third Creeks at 60 sq. mi. The intent was to implement the methodology as developed thus far without additional testing, thus serving as "validation basins". Both basins introduced new conditions, such as steep terrain, heavy regulation, ongoing development, very flat terrain with complex flow paths, and significant storm drain systems. These learnings helped shape this revised final draft (July 2026).

The District is implementing the Flood Risk Program to support a data-driven decision-making process that has numerous future applications. Examples of future applications include objectively prioritizing capital improvement projects based on effectiveness and cost-efficiency, assisting with maintenance priorities, informing the flood warning program, alternative hydrology approaches such as stochastic storm transposition, and many more. Additionally, data and models developed for this purpose can also assist in planning and floodplain management applications.

Core to these desired end uses is having a consistent approach to defensibly derive seamless, watershed-wide coverage of depth, velocity, and flood force estimates due to both pluvial flooding (localized ponding from rainfall) and fluvial flooding (concentrated flow in drainages greater than 130 acres for the purpose of this Program). Additional details on the Flood Risk Program can be found in the *MHFD Flood Risk Implementation Plan* (7/6/2026, Michael Baker International).

1.2 Intended Use

Studies performed within the District using 2D RoM methods are expected to follow this guidance unless watershed-specific conditions or study objectives warrant alternative approaches. Any alternative approach must be reviewed and approved by the District.

This document is intended to be used by consulting engineers, contractors, and technical staff responsible for developing, reviewing, or managing 2D RoM studies on behalf of MHFD. It provides a common technical framework for model development, calibration, documentation, and quality control (QC) to promote consistency across studies, arriving at standardized end products as well as improving program implementation efficiency. The guidance also supports reviewers by clearly defining expectations, assumptions, and minimum standards for submittals. This reduces ambiguity and the need for iterative clarification during review cycles.

The guidance supports the broader goals of the MHFD Flood Risk program by promoting defensible, repeatable, and transparent modeling practices that enable the consistent comparison of flood hazards and risk across watersheds. While the Federal Emergency Management Agency's (FEMA) Guidance and Standards for Flood Risk Analysis and Mapping Activities (G&S) provide a flexible national framework that allows for engineering judgment across a wide range of physiographic and development conditions (covered in Section 14), this document seeks to standardize key elements of the modeling approach for a more localized application within the District's predominantly urban environment. By doing so, the guidance helps ensure that Flood Risk products developed across different studies over time are technically comparable and suitable for long-term portfolio planning, communication, and equitable risk-based decision making.

1.3 Applicability

This guidance document primarily focuses on the use of HEC-RAS 2D, the Program's default software. However, InfoWorks ICM may also be used within the District's Flood Risk Program where it is already established and preferred by a local sponsor. The principles presented in this document generally apply to both HEC-RAS and InfoWorks ICM for 2D RoM H&H modeling studies within the District.

While this guidance is based on extensive testing and evaluation, the recommendations have been developed from a limited number of watersheds and users. As the Program advances, this guidance will continue to evolve based on experience gained across additional watersheds, feedback from local governments and practitioners, lessons learned during implementation, and ongoing efforts to improve usability, consistency, and effectiveness.

2 Key Elements Summary Table

This section provides a high-level summary of the key modeling requirements, preferred practices, and recommendations described throughout this guidance document. Table 1 is intended to function as a condensed reference that highlights where the District has established requirements, preferences, or guidance that extend beyond or further refine existing FEMA G&S. It does not fully capture important aspects of the detailed discussions in subsequent sections but serves to orient the reader to the major technical elements that govern the 2D RoM modeling studies performed for the MHFD Flood Risk program. The hierarchy field (third column) in Table 1 serves to indicate the type of influence or compliance for each aspect, as defined in the footnote below the table. Some aspects are standards to be complied with at all times, others are recommended processes for efficiency, and some are recommended considerations in areas of subjective judgment. Table 1 should be consulted in conjunction with the following sections of this guidance document to understand where there are strict requirements versus flexibility in guidelines.

The structure for Table 1 generally follows the structure of the guidance document, with each row corresponding to a major topic addressed in later sections, allowing users to quickly understand how individual modeling components fit within the overall framework and where to find additional detail. The "Current FEMA Guidance" column refers to the current status of FEMA governing applications for a given topic.

Table 1. Summary of Key Elements in this Guidance Document

Category	Element	Hier-archy	Key Components	Current FEMA Guidance Status
Organization	Folder Structure	R	Standardized folder organization for consistent submittals and model linking	Partially – different structure
Model Setup	Naming Convention	R	Standardized for files, plans, inputs, outputs	Only spatial schemas
	Model Descriptions	R	Standard documentation in windows for the project, unsteady flow, and plans.	Absent, but usually customized in project team guidance
	Software Requirements	R	New RoM models must be developed in either InfoWorks ICM or HEC-RAS 7.0, depending on agreements with the local sponsor. Currently there is no process for using RAS 2025, but this will be revisited for future releases (e.g., RAS 2027).	Nothing on RAS versions or capabilities. FEMA accommodates many options and is moving toward RAS2027 (or later) for Future of Flood risk Data (FFRD).
	Domain Delineation	C	Size, overlaps, and influences (gages, community boundaries, development). The study area is defined as the region unaffected by boundary condition influences, within which model results are considered to have an acceptable level of confidence.	No – usually directed by Project Manager and Subject Matter Experts.
	Coordinate System and Datums	R	Horizontal: NAD_1983 (2011) EPSG:6428. Projection: State Plane Colorado Central FIPS 0502 Feet	Not Applicable

Category	Element	Hier-archy	Key Components	Current FEMA Guidance Status
			Vertical: NAVD88	
	Terrain	R	QL1/QL2 only. Bathymetry only if Light Detection and Ranging (LiDAR) data are inadequate (infrequent).	QL1/2 clearly defined. Minimal guidance on bathy. None on bldgs.
Input Layers	Land Cover	P	Denver Regional Council of Governments (DRCOG) and supplement with machine learning as needed. National Land Cover Database (NLCD) is appropriate for rural areas (with roads and buildings burned in) where DRCOG coverage may be absent. Buildings represented with high Manning's values.	None
	Soils	P	Soil Survey Geographic Database (SSURGO), otherwise GSSURGO	None
Hydrology	Design Storms (synthetic events)	P	National Oceanic and Atmospheric Administration (NOAA) and the methodology outlined in MHFD's Urban Storm Drainage Criteria Manual (USDCM) Recurrence intervals: six regulatory and 13 for Flood Risk	Just recurrence intervals with minimal content on distributions. Not CUHP
	Depth Area Reduction	R	Handled via the methodology outlined in MHFD's USDCM	Briefly discussed but is further expanded upon in this guidance.
	Infiltration	G	Curve number (CN) for single peak and <48 hrs. Green and Ampt (G&A) for multipeak or >48 hrs. Apply impervious percentage.	None
	Conservative Assumptions	R	50-year design storm event run for at least 7-day duration to allow for full drainage and inadvertent storage to be filled	None
	Calibration/ Validation Sources	J	District Gage Adjusted Radar Rainfall (GARR), Multi-Radar/Multi-Sensor (MRMS), Analysis of Record for Calibration (AORC), Log Pearson type III (LPIII) Gage, Regional Regression Equations (RREs), historical flood imagery or records, and previous detailed studies	Generic principles, without specifics or practical steps
	Regulation, Reservoirs, and Detention	P	Discharge based on operations manual and published rating curves. Assume normal pool elevation and bathymetry for all design storms. Extract information from prior models and as-builts, if available.	Varies but there is specific guidance for dams and reservoirs

Category	Element	Hierarchy	Key Components	Current FEMA Guidance Status
	Inflows and Basin Linking	J	Assume offset peaks when regulated and some statistical coincidence via joint probability if unregulated.	Briefly discussed without specifics
	Corridor Modeling	J	Trimmed corridor with scaled inflows and multiple scenarios	None
	External Boundary Conditions	G/J	Include boundary conditions (BCs) at channel outflows and also surrounding the buffered perimeter with a normal depth boundary condition (BC) to allow full unrestricted drainage. Set slope to match the channel or flow paths for all normal depth BCs. Inflow hydrographs linked to a digital storage system (DSS) to facilitate ready updates (not manual entry). Model linking and tie-ins with sufficient boundary overlap for a transition.	None – these are all best practices, but they are not specified in any guidance.
	Internal Boundary Conditions	P	Used to simulate base flows or flow additions potentially scaled based on a joint probability analysis. Must cover multiple cells for stability (with correct enforcement between banks).	None – these are all best practices, but they are not specified in any guidance.
Geometry Construction	Mesh Enforcement	G	50-foot nominal in urban areas and 100- to 200-foot in undeveloped areas. Refinements between 25 and 100 feet, with reduced cells at 25 to 100 feet along breaklines and structures.	Mostly absent, though relative sizes (small, medium, coarse) are specified in Base Level Engineering (BLE) Levels.
	Underground Stormwater Infrastructure	G	Pre-process data prior to modeling. Model as pipes in HEC-RAS 6.7 or more recent. All inlets must be included if the data is available. Terrain analytics will evaluate sumps to anticipate relative influence on drainage. Additional locations may be added on a per-study basis where arbitrary risk is being applied due to absence of a stormwater system.	FEMA does not account for impacts from underground stormwater systems (assumes at capacity) in regulatory studies. This may change for FFRD and is presently being piloted.
	Hydraulic Structures	G	Storage Area/2D (SA/2D) connections must be utilized based on survey data or existing Flood Hazard Area Delineation (FHAD) data. Where no data are available, structure information may be approximated in coordination with the District and local governments. In undeveloped regions where no structure data are available, structures can be	The presence of modeled structures is addressed in scoped BLE Level, but their representation is open-ended.

Category	Element	Hier-archy	Key Components	Current FEMA Guidance Status
			represented through terrain modifications of approximate opening size and length.	
	Solutions Equations and Parameters	C	Trial runs use Diffusion Wave Equation (DWE) with max courant of 3, later decreased to 1 for troubleshooting. Calibration and final runs can use shallow water equations (SWE) with a courant of 1 and a minimum timestep as low as 0.5 second.	None – too software specific.
	Calibration	P/G	Historical calibration to the observed record with two historical events, ideally with one of higher magnitude (20-100 year) and one of lower (2-10-year). Additional historical events may be used if necessary. Time series volumetric calibration while matching peak flows. Apply statistical metrics and volume check. Reference historic imagery and high water marks where available.	Some of the concepts are defined, but not the criteria.
Sims	Model Stability and Errors	R	Total volume accounting errors should not exceed 2%. Water surface elevation (WSE) errors >0.2 foot along the main channel require justification.	None – too software specific.
Simulations and Output	Modeling and Mapping Thresholds and Filters	P	Map connected fluvial flooding to a depth of 0.1 foot for areas of 130 acres (0.20 sq. mi.). Map pluvial flooding to a depth of 0.1 foot for >0.5 acre when the average depth is greater than 0.5 foot or max depth exceeds 1 foot. Designate as separate “pluvial zone.”	Fluvial flooding is the focus and cuts off at 1 sq. mi. Ponded pluvial is generally not mapped and often not provided.

R = *Required compliance*: standardized criteria are explicitly defined and must be adhered to for an acceptable submittal regardless of the process used to meet the criteria.

P = *Prescriptive practice*: a process is clearly outlined with particular objectives. The given process is suggested for consistent results, but alternate processes are acceptable as long as the associated criteria are still achieved.

G = *Guidance or guidelines*: the objectives are clear, but the process to achieve them is less rigid and there may be times when all objectives cannot be equally met.

J = *Judgment*: while some guardrails are presented, the outcome/implementation is subjective to the project team’s discernment. These matters generally pertain to subjectivity in areas of technical defensibility.

C = *Consideration*: factors that ought to be considered in the decision-making process. These often are recommendations for a more efficient or stable execution, but don’t necessarily reflect on how sound an approach might be from a technical perspective.

3 Model Setup and Documentation

This chapter outlines steps for organizing folder structures, naming conventions, project and plan descriptions, software requirements, and domain delineation. Standardizing these elements leads to projects that are easy to navigate, update, and review, which facilitates collaboration and automation.

3.1 Modeling Folder Structure

Figure 1 provides the modeling folder structure that should be followed for all 2D RoM studies within the District.

```
Model_Name/
│
├─ HEC-RAS_Model_Files_VerXXX/
├─ Reference/
│  ├─ Calibration/
│  │  ├─ Calibration Report
│  │  ├─ Stream Gage Analysis
│  │  └─ Calibration Excel Workbook
│  │
│  ├─ Inflows/
│  │  ├─ Inflow DSS Files
│  │  ├─ Inflow Report
│  │  └─ Inflow Excel Workbooks
│  │
│  ├─ Land_Classification/
│  │  ├─ LandCover.tif (or .shp)
│  │  ├─ LandCover.hdf
│  │  ├─ Soils.tif
│  │  ├─ Soils.hdf
│  │  ├─ ManningsN.tif
│  │  ├─ ManningsN.hdf
│  │  ├─ ManningsN_DepthWeighted.tif
│  │  └─ ManningsN_DepthWeighted.hdf
│  │
│  ├─ Precipitation/
│  │  ├─ Design Storm Hydrographs (Excel or DSS)
│  │  ├─ Calibration Storms (Excel or DSS)
│  │  └─ Precipitation Report
│  │
│  ├─ Storm_Sewers/
│  │  ├─ Stormwater System Shapefiles
│  │  └─ Stormwater System Notes
│  │
│  ├─ Spatial_Reference/
│  │  └─ Spatial_Reference.prj
│  │
│  ├─ Terrain/
│  │  ├─ Terrain.tif
│  │  ├─ Terrain.hdf
│  │  └─ HEC-RAS Terrain files (.vrt, .hdf, clones)
│  │
```

```
│     └─ DetentionPonds_RatingCurves/
│          ├─ Rating Curve PDFs
│          └─ Rating Curve Spreadsheet (with source documentation)
└─ Supplemental_Information/
        ├─ Canals
        ├─ Watershed
        ├─ Metadata and Data Source Inventory
        ├─ Study Area Boundary Shapefile
        └─ Domain and Subbasin Boundary Shapefile
```

Figure 1. Modeling Folder Structure

Additional files may be included under appropriate parent folders where necessary and a supplemental folder may be added to include all additional information important to future users of the model. It is preferred to maintain the file structure defined in the template without modification, adding additional folders only as needed or when appropriate. Folders without data should not be removed; instead, include a README indicating that "this folder is intended to be empty. It is also essential to avoid "file name too long" error that often occurs in deeply nested folder structures. It is encouraged to adopt concise naming conventions following the template below in order to avoid such obstacles. For example, the base terrain should be simply named, "Terrain.tif" to reduce the number of characters.

3.1.1 HEC-RAS File Structure and Organization

HEC-RAS projects while in development will often result in large numbers of unused HEC-RAS files that were created during intermediate processes. The most common examples of such files are excess geometry files or interim result files produced during model development and calibration. Final submittals should remove all excess HEC-RAS files that are no longer relevant to or utilized in the final model. The most common HEC-RAS file extensions are listed below in Table 2. A full list of HEC-RAS file extensions and their use cases can be found in the HEC-RAS 7.0 User Manual.

Table 2. Common HEC-RAS File Extensions

.prj	Project file
.p##	Plan file
.p##.hdf	Result file associated with similarly numbered plan file
.x##	Run file for unsteady flow plans
.g##	Geometry file
.g##.hdf	HDF file corresponding to a specific geometry file
.u##	Unsteady flow plan
.b##	Boundary condition file executed for each plan
.bco##	Unsteady flow log output file for project
.ic##	Initial condition file for each unsteady flow plan executed
.p##.rst	Restart file associated with a model plan/run

The best practice for determining if intermediate files are no longer relevant or unused in the current model is to go through each extension type and the location of such files in the current model. For example, when reviewing .u## files which are unsteady flow editor files it is recommended to review the final submittal unsteady flow files included within the HEC-RAS model and delete all numbers that are no longer represented in the submittal from the model folder. Similarly reviewing result file numbers can significantly reduce file size removing .p##.hdf files that are not included in the final results but may not have been removed from the parent folder.

3.2 Naming Convention

Models developed for the District shall follow a standardized naming convention to promote consistency, ease of sorting, and long-term file management. Model names shall include "FR" to indicate it is part of the Flood Risk Program, the year in which the study was initiated (by the kickoff meeting date), followed by the name of the primary flooding source being evaluated. Note that "Creek" should be abbreviated to "Ck" to reduce file name lengths.

The required format is FR_####_CreekName. For example: FR_2026_SandCk.

The creek name shall correspond to the basin of the primary flooding source being mapped. Where a study is subdivided into multiple domains (e.g., Upper, Middle, Lower), the domain descriptor may be appended to the name as needed for clarity, provided the base naming convention is preserved. Within the MHFD there are a significant number of unnamed creeks. If the primary flooding source of the study area is an unnamed creek the required naming format is "Municipality_NNC###". The numbers following "NNC" must come from the MHFD GIS database "Streams" layer and "mhfd_code_stream" attribute associated with the primary flooding source.

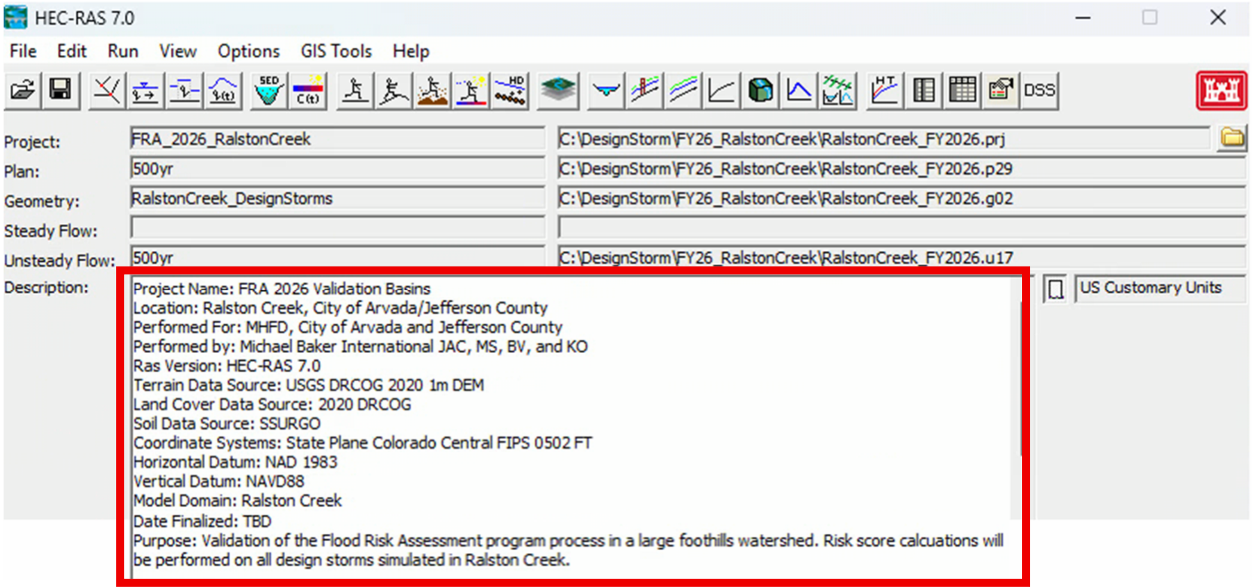
3.3 Standardized Descriptions

The descriptions included in the editor windows (project, plan, unsteady flow, geometry) must be updated to contain pertinent information to the study and watershed. This section outlines the minimum requirements for all descriptions.

3.3.1 Project Description

Project descriptions must contain at least the following information (Figure 2):

- Project name and number (if applicable)
- Location (Local Government(s) and flooding source)
- Performed for MHFD and Local Government(s) (if applicable)
- Performed by (Company) and Modeler(s) initials
- RAS version used
- Model domain
- Terrain data source (LiDAR Capture Year and any sources used for bathymetry)
- Landcover data source
- Soil data source
- Geographic Coordinate system; Horizontal Datum; Vertical Datum
- Date finalized



Project: FRA_2026_RalstonCreek C:\DesignStorm\FY26_RalstonCreek\RalstonCreek_FY2026.prj
Plan: 500yr C:\DesignStorm\FY26_RalstonCreek\RalstonCreek_FY2026.p29
Geometry: RalstonCreek_DesignStorms C:\DesignStorm\FY26_RalstonCreek\RalstonCreek_FY2026.g02
Steady Flow:
Unsteady Flow: 500yr C:\DesignStorm\FY26_RalstonCreek\RalstonCreek_FY2026.u17
Description:
Project Name: FRA 2026 Validation Basins
Location: Ralston Creek, City of Arvada/Jefferson County
Performed For: MHFD, City of Arvada and Jefferson County
Performed by: Michael Baker International JAC, MS, BV, and KO
Ras Version: HEC-RAS 7.0
Terrain Data Source: USGS DRCOG 2020 1m DEM
Land Cover Data Source: 2020 DRCOG
Soil Data Source: SSURGO
Coordinate Systems: State Plane Colorado Central FIPS 0502 FT
Horizontal Datum: NAD 1983
Vertical Datum: NAVD88
Model Domain: Ralston Creek
Date Finalized: TBD
Purpose: Validation of the Flood Risk Assessment program process in a large foothills watershed. Risk score calcuations will be performed on all design storms simulated in Ralston Creek.

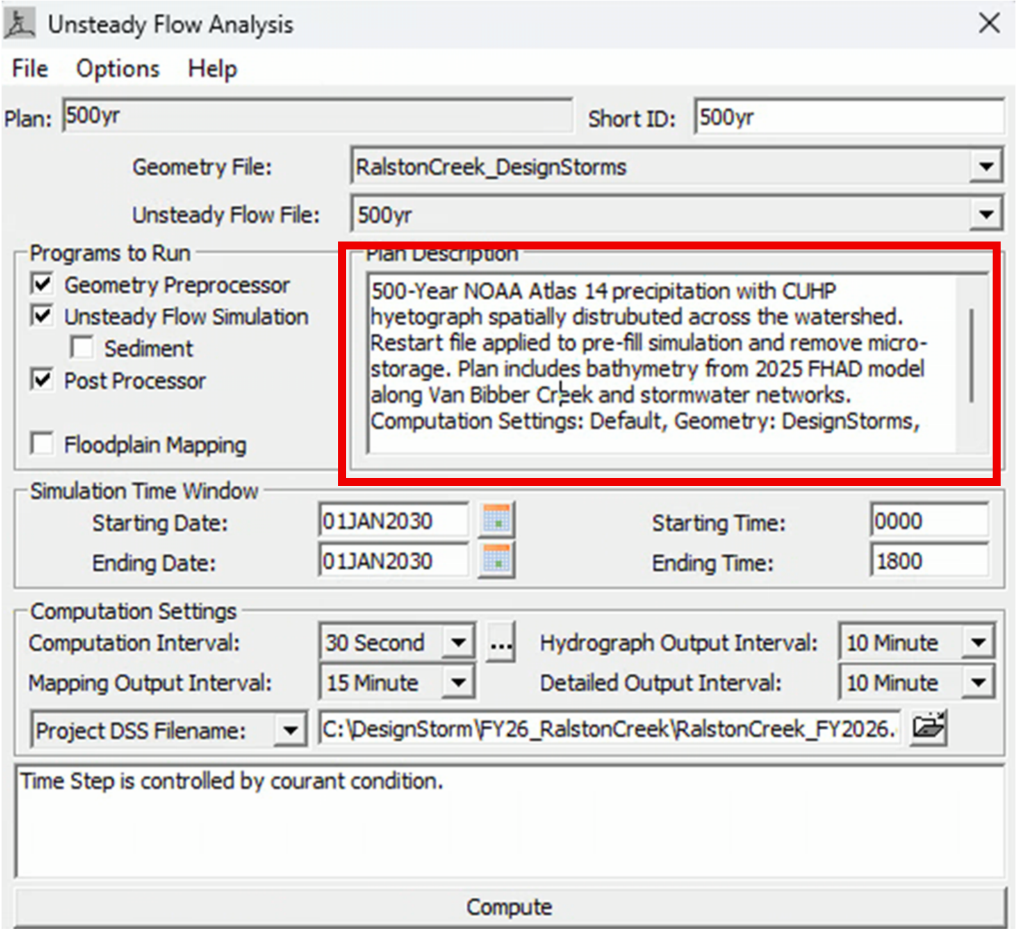
Figure 2. Example of HEC-RAS Project Description

3.3.2 Plan Nomenclature and Description

Plan names for design storms should follow these examples:1yr, 10yr, 100yr, 1000yr, etc.

If a plan is run for supplemental purposes, a reasonable title should follow the above format including an underscore. For example, “100yr_GA” for G&A infiltration. For all calibration events the plan name should follow the format of “June2020_Calib” or “June2020_Valid.”

Plan descriptions should include a brief description of the simulation run and purpose. List any assumptions or unique attributes specific to the individual plan. An example plan description can be seen in Figure 3. Common inclusions in plan descriptions are considerations and exceptions for instability, changes in hydrological methods, changes in boundary conditions, changes in computational settings, etc. If there are no changes to computational settings please include, “Computation Settings: Default” Plan descriptions should also include the geometry file and unsteady flow file used as a back-up for future use cases.



Plan: 500yr
Short ID: 500yr
Geometry File: RalstonCreek_DesignStorms
Unsteady Flow File: 500yr

Programs to Run
☑ Geometry Preprocessor
☑ Unsteady Flow Simulation
☐ Sediment
☑ Post Processor
☐ Floodplain Mapping

Plan Description
500-Year NOAA Atlas 14 precipitation with CUHP hyetograph spatially distrubuted across the watershed. Restart file applied to pre-fill simulation and remove micro-storage. Plan includes bathymetry from 2025 FHAD model along Van Bibber Creek and stormwater networks. Computation Settings: Default, Geometry: DesignStorms,

Simulation Time Window
Starting Date: 01JAN2030
Ending Date: 01JAN2030
Starting Time: 0000
Ending Time: 1800

Computation Settings
Computation Interval: 30 Second
Hydrograph Output Interval: 10 Minute
Mapping Output Interval: 15 Minute
Detailed Output Interval: 10 Minute
Project DSS Filename: C:\DesignStorm\FY26_RalstonCreek\RalstonCreek_FY2026.

Time Step is controlled by courant condition.

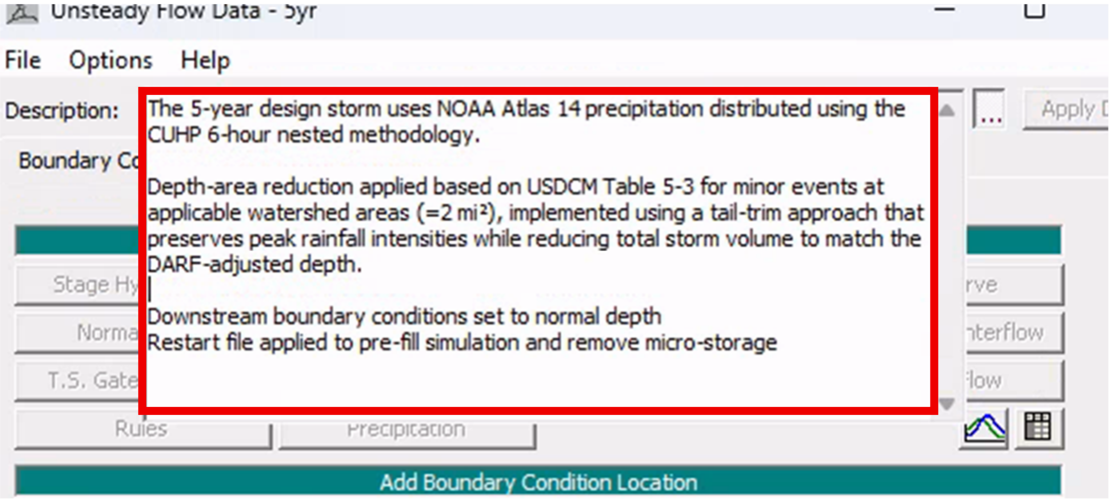
Figure 3. Example Plan Description

3.3.3 Unsteady Flow Editor Description

Unsteady flow editor files will follow the same nomenclature as design storms (1yr, 10yr, 100yr, 1000yr, etc.) and calibration events (e.g., “June2020_Calib” or “June2020_Valid”). If an unsteady flow file is required for supplemental purposes, a reasonable title should follow the above format including an underscore. For example, “100yr_NoRestart” for an unsteady flow file omitting the required restart file.

Unsteady flow descriptions should include all sources and assumptions specific to a given file’s hydrology (Figure 4). These include the following parameters:

- Precipitation source and timestep interval
- Depth area reduction factor applied (list 0 if none is applied)
- Description of inflow sources (if applicable)
- Add recurrence intervals and precipitation duration for design events.
- Description and assumptions made for all boundary conditions used
- Description and assumptions for all initial conditions set including sources



The 5-year design storm uses NOAA Atlas 14 precipitation distributed using the CUHP 6-hour nested methodology.

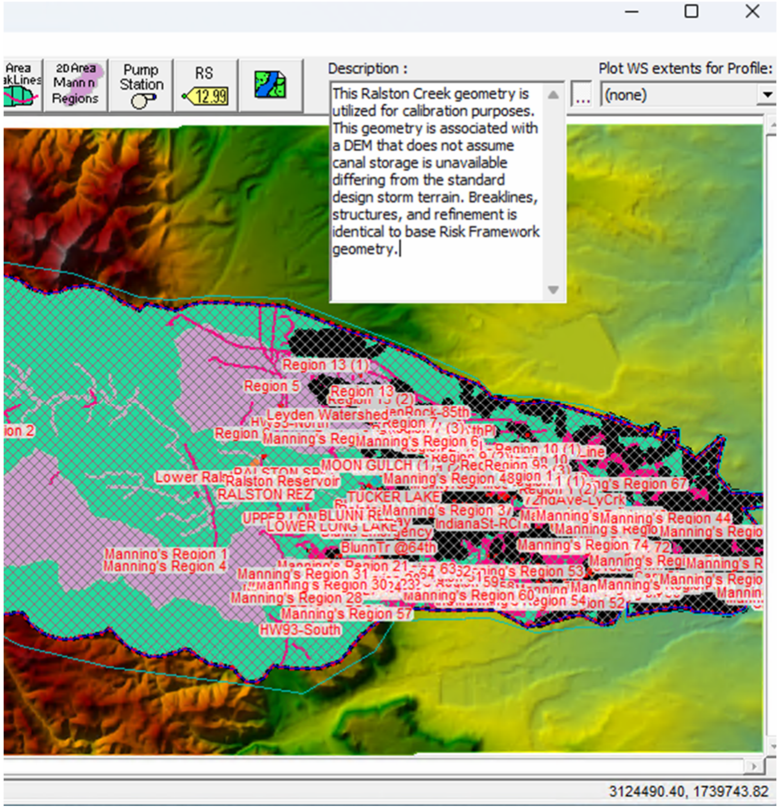
Depth-area reduction applied based on USDCM Table 5-3 for minor events at applicable watershed areas (=2 mi²), implemented using a tail-trim approach that preserves peak rainfall intensities while reducing total storm volume to match the DARF-adjusted depth.

Downstream boundary conditions set to normal depth
Restart file applied to pre-fill simulation and remove micro-storage

Figure 4. Example Unsteady Flow Editor Description

3.3.4 Geometry Description

The geometry description should be populated for each individual geometry associated with the model. For the majority of cases models should be delivered with one clear geometry, but for sensitivity analysis, additional considerations, or calibration purposes such as assumptions made around canals there may be multiple geometry files. Geometry descriptions should include a brief description of the geometry as well as any assumptions that may assist the end user.

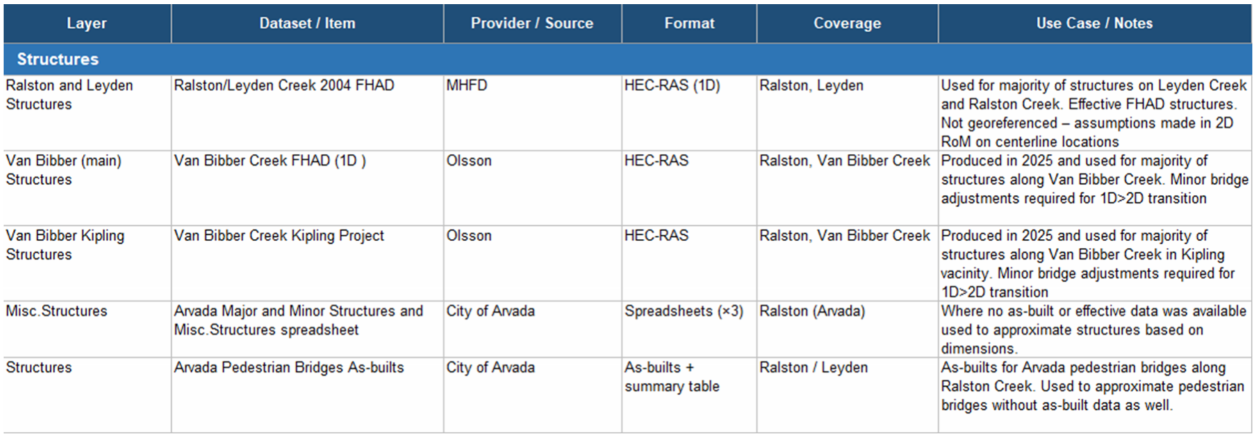


This Ralston Creek geometry is utilized for calibration purposes. This geometry is associated with a DEM that does not assume canal storage is unavailable differing from the standard design storm terrain. Breaklines, structures, and refinement is identical to base Risk Framework geometry.

Figure 5: Example Geometry Description

3.4 Metadata and Data Source Inventory

Project submittals shall include metadata and a data source inventory documenting each dataset used to develop the model and supporting products. At a minimum, the inventory shall identify the layer or application for which the dataset was used; for example: structures, land cover, bathymetry, soils, stormwater network, hydrology, historical precipitation, buildings, roads, and prior HEC-RAS models. For each dataset, the submittal shall document the dataset name, provider or source, original format, geographic area of coverage, intended use within the study, and any relevant notes regarding limitations, assumptions, or applicability to model development and review.



Layer	Dataset / Item	Provider / Source	Format	Coverage	Use Case / Notes
Structures					
Ralston and Leyden Structures	Ralston/Leyden Creek 2004 FHAD	MHFD	HEC-RAS (1D)	Ralston, Leyden	Used for majority of structures on Leyden Creek and Ralston Creek. Effective FHAD structures. Not georeferenced – assumptions made in 2D RoM on centerline locations
Van Bibber (main) Structures	Van Bibber Creek FHAD (1D)	Olsson	HEC-RAS	Ralston, Van Bibber Creek	Produced in 2025 and used for majority of structures along Van Bibber Creek. Minor bridge adjustments required for 1D>2D transition
Van Bibber Kipling Structures	Van Bibber Creek Kipling Project	Olsson	HEC-RAS	Ralston, Van Bibber Creek	Produced in 2025 and used for majority of structures along Van Bibber Creek in Kipling vacinity. Minor bridge adjustments required for 1D>2D transition
Misc.Structures	Arvada Major and Minor Structures and Misc.Structures spreadsheet	City of Arvada	Spreadsheets (×3)	Ralston (Arvada)	Where no as-built or effective data was available used to approximate structures based on dimensions.
Structures	Arvada Pedestrian Bridges As-builts	City of Arvada	As-builts + summary table	Ralston / Leyden	As-builts for Arvada pedestrian bridges along Ralston Creek. Used to approximate pedestrian bridges without as-built data as well.

Figure 6: Example of Metadata Catalog

For individual datasets additional metadata may be helpful for transparency on delivery. Pipe networks for example will often require their own metadata for reviewers or end users of the model to understand attributes and processing procedures that were performed. As part of the output shapefiles should be included along with the project metadata catalog. At a minimum the following shapefiles should be included but additional model input layers should be included if applicable.

Table 3. Required Supplemental Output Shapefiles

Shapefile	Note	Shapefile Type	Required Fields
Structures	shapefile identifying the location of each structure and can serve as an index to source information	Point	name, type, size, source, notes
Stream Centerlines	Centerline shapefile for all streamlines within a watershed.	Polyline	name, watershed
Stormwater Nodes	Point shapefile of the modeled Nodes.	Point	All HECRAS generated fields + source flag fields (zInvert, zGround etc.) + supplemental (Owner, Notes etc.).
Stormwater Conduits	Polyline shapefile of the modeled Conduits.	Polyline	All HECRAS generated fields + source flag fields (zShape, zMaterial, zDiameter etc.) + supplemental (Install yr, active status, Notes etc.).

Shapefile	Note	Shapefile Type	Required Fields
Buildings	Building shapefile used for land cover layer.	Polygon	source
Terrain Modifications	Terrain modification shapefile used for channels, culvert inlet/outlets, or any additional terrain adjustments.	Multiple (lines, polygons)	purpose, notes
Structure Centerlines (From HEC-RAS)	Structure centerlines from SA/2D connections.	Polyline	Default HEC-RAS fields

3.5 Software Requirements

The list of currently accepted software by FEMA is listed here: Hydraulic Numerical Models Meeting the Minimum Requirement of National Flood Insurance Program | FEMA.gov. FEMA G&S does not prescribe specific HEC-RAS version requirements and allow flexibility to accommodate a range of study types, software capabilities, and evolving modeling practices. For consistency within the District's Flood Risk program, 2D RoM models shall be developed and maintained using HEC-RAS version 7.0 or InfoWorks ICM. This requirement applies to all new studies as well as any previously developed models that are updated, revised, or relied upon for current analyses.

Because representation of underground stormwater infrastructure is a required component of District Flood Risk studies, earlier versions of HEC-RAS are not permitted due to known limitations affecting pipe network modeling and stability. As a result, legacy models developed in earlier HEC-RAS versions must be upgraded to HEC-RAS 7.0 prior to use. The HEC-RAS version used for each study shall be clearly documented in the project description and model metadata.

InfoWorks ICM is also acceptable for the District's Flood Risk Program, and will be used when a precedent has already been established and/or it is preferred by local sponsor. At this time, ICM is the planned approach for most watersheds within the City of Boulder, City and County of Denver, and the Southeast Metro Stormwater Authority. The initially planned delineation of watersheds using ICM instead of HEC-RAS is included in the MHFD Flood Risk Program Implementation Plan. This guidance document is focused on the use of HEC-RAS 2D; however, the principles generally apply to the use of InfoWorks ICM for 2D RoM H&H modeling studies within the District, and there is a separate document detailing ICM-specific modeling guidance. Specific guidance may need to be developed for differences between modeling software; for example, how to create a restart file in ICM.

3.6 Rain-on-Mesh Domain and Study Area Delineation

RoM model domains should be delineated with consideration for natural divides, efficient run times, model stability, and rendering times. It should be noted that riverine domains will be derived as pseudo-steady state (PSS) models from the final RoM models, as discussed in Section 11.

Model domains should generally be a minimum of 7 sq. mi. for urbanized settings and may be up to 100 sq. mi. if largely rural or undeveloped. For larger urban areas models should be kept to less than 25 sq. mi due to the increased run times associated with large pipe networks and structures as SA/2D connections. It is recommended that models target run times less than 30 hours for design storms. This tends to be driven largely by the pipe systems, followed by the number of hydraulic structures – and is less a function of total model domain size. Longer run times might be encountered when simulating longer historical events during calibration. If model run times are prohibitive, model settings and stability should first be examined before further dividing the watershed (Section 9.4). If adjustments to model settings are insufficient to achieve reasonable model run and load times further division of the watershed into subdomains may be required.

The subdomains should be delineated using the digital elevation model (DEM) that will be used as the modeling terrain to ensure that the subdomains follow the true watershed boundaries and edge-match with neighboring subdomains. Subbasins must follow the same guidance as model domains. Subdomains should be unaffected by boundary condition influences and model results must not be impacted by the boundary location.

The domain boundary(s) should be smoothed and simplified to minimize the number of vertices and jagged shapes that can cause cell errors in HEC-RAS. The recommended starting point for domain delineation smoothing is 25 feet with Polynomial Approximation with Exponential Kernal (PAEK) smoothing algorithm in GIS. This is in alignment with the floodplain mapping recommendation in FEMA 2D BLE Best Practices published October 2025. Deviation is expected for watershed boundaries to follow complex, highly detailed urban areas where a smoothing tolerance may not be applicable or needs to be decreased. Where flow will exit the study area, domain boundaries should be edited to be perpendicular to flow and extend past anticipated boundary condition affects. Watershed domain boundaries that fall within highly developed areas of the District, where the watershed boundary is not clear, should be buffered to maintain an overlap with the neighboring domain to fully capture high ground and account for all structures in the risk assessment. Where watershed boundaries are not required to be buffered along boundaries clearly defined by the DEM vertices should be identical between adjacent domains.

RoM study area boundaries must also be clearly defined with a polygon shapefile included in the “supplemental” folder included with model submission. The study area boundary is defined as the useable region unaffected by boundary condition influences, within which model results are considered to have an acceptable level of confidence. The inclusion of a designated study area boundary is to assist in adjacent area tie-ins and support defining the governing results for a given area.

3.6.1 Inter-Domain Transitions and Model Tie-Ins

Inter-domain flows are used to support transitions or tie-ins between upstream and downstream model domains when a larger watershed is divided into sequential study areas. This section applies specifically to upstream–downstream transitions and does not address adjacent or side-by-side model domains, which are discussed separately in Section 3.6.2.

At potential inflow locations between study areas, model domains must overlap. Overlapping and flow-exchange locations should occur at hydraulically stable sections of the system to reduce uncertainty when determining inflows and outflows between domains. Transition locations should avoid splitting community boundaries, highly developed areas, wide floodplains, or locations with complex hydraulic behavior that could complicate the transfer of flows. Transition breaks are encouraged at stream gage locations, dam outlets, or other locations where observed data are available, as these locations can serve as effective reset points for hydrograph timing and calibration.

Once general transition locations have been identified, the overlapping area between upstream and downstream domains shall be used to facilitate flow exchange while avoiding duplication of hazard and risk products. Therefore, the downstream portion of the transition zone is excluded from the upstream model, and the upstream portion of the transition zone is excluded from the downstream model. Each portion of the watershed within the transition area is thus represented in only one model for the purpose of generating final flood hazard and risk outputs.

The upstream domain should contain a reference line within the overlapping area that can be directly translated into an inflow boundary condition line in the downstream model. The reference line should additionally be enforced as a breakline. Areas of overlap and inflow boundary condition locations must be sufficiently upstream of the selected study area such that the distribution of the inflow boundary condition does not influence results within the study area. Any study area or domain boundaries within the overlapping zone should be enforced as breaklines or refinement regions to align cells with the perimeters. The WSEs within the overlapping region should be reviewed to confirm reasonable agreement between models, and WSEs in the overlap area should be verified to be within a tolerance of 0.5 foot to support smooth transitions during floodplain mapping. Domain overlaps and transition logic should be documented to clearly demonstrate continuity between models.

See Section 6.4 for more details around inter-basin flows and model linking.

3.6.2 Overlapping Domains in Flat Terrain

In flat or weakly defined terrain, natural drainage divides may be uncertain, poorly defined, or sensitive to small changes in terrain, land development, or boundary condition placement. In these settings, strict predefined watershed delineation can result in incomplete drainage representation which will potentially produce inaccurate flood risk scores. To address these conditions, intentional domain overlaps may be required to ensure full hydraulic coverage and realistic drainage behavior.

Where intentional overlaps are used, adjacent or sequential model domains are extended beyond the apparent divide so that uncertain flow paths are captured in at least one model domain. Normal depth external boundary conditions are applied along the overlapping perimeter to allow water to freely drain out of each domain and to prevent artificial backwater effects at the study boundary.

Portions of the watershed within these intentional overlap areas will be simulated in more than one model domain. To avoid duplication of hazard and risk products, the final flood hazard and risk outputs for overlapping areas shall first be clipped to the useable study boundary (if portions of the domain are invalid, such as buffered regions). The final result will then be derived by taking the maximum value from the multiple model simulations at each location. This approach ensures conservative representation of flooding while maintaining continuity across domains and avoiding gaps caused by uncertain drainage divides.

This methodology allows flat terrain areas to be modeled robustly without relying on a single, potentially incorrect delineation of flow direction. An example of intentional domain expansion and overlap used to address uncertain flow paths in flat terrain is shown in Figure 7, which illustrates expanded adjacent model extents and the resulting overlap zones.

Figure 7. Example Need for Domain Expansion and Overlap due to Uncertain Flow Paths

4 Terrain

Accurate terrain representation is critical in 2D hydraulic modeling, as it directly influences flow paths, routing, ponding, and flood extents. This chapter details the requirements for terrain resolution and quality, the integration of bathymetric and surveyed data, and the proper representation of buildings and other features. Standardizing terrain processing aims to ensure that models within the District meet vertical accuracy specifications and consistently represent ground and structure elevations.

4.1 Resolution and Quality

High-resolution terrain data (approximately 1-meter or finer) are essential for accurate urban 2D RoM flood modeling. Urban corridors contain numerous small features such as bridges, culverts, curbs, buildings, weirs, and diversion structures that strongly influence water flow and ponding. Coarser terrain grids (5 to 10 meters or larger) tend to smooth out these critical details which can result in unrealistic flow paths or pooled water in the model. The default terrain selection for the Flood Risk Program is the 1-meter DEM collected by USGS and provided by DRCOG. To search for publicly available DEM datasets please refer to: https://apps.nationalmap.gov/downloader/. **Prior to selecting a terrain for model production outside of the standard DRCOG DEM, consultation and approval from the MHFD is required.**

Research conducted by Fewtrell et al. (2011) compared a 0.5-meter grid to a 5-meter grid and found the finer resolution notably improved model accuracy by capturing road camber and curbs that confine runoff. Similarly, Lee et al. (2025) found that flood extent predictions became much less accurate when DEM resolution was coarser than 7-meters due to the loss of narrow road and drainage details that channelize urban runoff. Therefore, a 1-meter DEM should be used. In the event that a finer resolution is available (e.g., 0.5-meter), the option of resampling to one meter should be discussed with MHFD and the local sponsor. Having very fine DEMs without changing the hydraulic mesh offers limited benefits but increases field sizes and data processing. Maintaining a consistent 1-meter resolution allows for future updates to have a 1:1 comparison.

If the highest resolution terrain is not the most recent, then development changes in the watershed should be evaluated to determine the relative tradeoff between resolution and recency. For example, if a new DEM was processed in 2027 at 1-meter, it would likely provide better analysis than a 0.5-meter DEM from 2020, but perhaps not if at a 3-meter resolution. If the resolution is greater than 1-meter, its potential to artificially widen the floodplain should be inspected and documented. All ground and structure surveys used in terrain development should be certified with a formal assessment conducted. FEMA SID 42 pertains to this and ensures that depth and analysis grids remain consistent with the terrain and bathymetry sources used in the engineering models.

4.2 Coordinate System

Consistency in spatial reference is essential for integrated workflows between modeling, outputs, and delivered products. Spatial data produced in association with the Flood Risk program shall follow the coordination system listed below. This includes foundational data layers, the H&H models, exported gridded data, and resulting products.

Projected Coordinate System: NAD 1983 (2011) State Plane Colorado Central FIPS 0502 (EPSG:6428)

Projection: Lambert Conformal Conic (2SP)

Units: US Survey Feet

Vertical Datum: North American Vertical Datum of 1988 (NAVD88)

4.3 Channel Bathymetry

Most of the streams in the District will not benefit from modifying the DEM with channel bathymetry, as LiDAR often adequately represents smaller drainages or shallower clear streams. Channel bathymetry should be included where reliable data already exist for conditions where the stream bottom is obscured due to incised channels, heavily vegetated streams, or deeper flows. In urbanized streams, the actual channel cross section is often deepened or confined by constructed walls, which can meaningfully influence geometry in conveying flow. Channel geometry might also be modified near structures, especially those prone to scour. It should be noted that the Flood Risk Program intends to utilize readily available information, but does not involve collecting field survey data.

If existing bathymetric data are available from field surveys, prior models, or other local sources, the data should first be reviewed to determine whether to incorporate into the terrain file in RAS Mapper. Recent LiDAR collections are often sufficient for most shallow or clear channels, such that transposing channel geometry from prior models may be erroneous, partly due to extended reaches of interpolation but largely because the stream geometry is dynamic and often changes over the years. In areas where bathymetric data might be useful, it ought to be collected recently, based on field survey, and incorporated only in select reaches as needed.

There may be select circumstances where the channel geometry is not captured even between banks above water, such as due to heavily vegetated corridors. In such instances, the DEM may be highly variable along the stream centerline with “divots” reflecting pockets where LiDAR penetrated the canopy, and “mounds” where LiDAR reflected off the canopy. In these cases, it can be helpful to modify the channel geometry with manual overrides, not to capture bathymetry below water, but to capture the bankfull flow corridor. This process is subjective and optional to make iterative adjustments to estimated parameters (width, depth, slope) to improve calibration as needed. Document assumptions and justify dimensions by comparing modeled results to known rating curves. RAS Mapper tools allow terrain modifications using trapezoidal sections; multiple reaches or stacked edits may be required to achieve realistic channel shapes.

4.4 Reservoir and Detention Facility Bathymetry

For larger storage facilities (reservoirs and detention), terrain modifications below the normal pool are often necessary to improve stability by increasing storage for rating curve and structure inlets. If there is no outlet structure, then no terrain modification or bathymetry will be necessary for storage as the elevation reflected in the DEM is assumed to be representative of the normal pool, unless otherwise documented.

While surveyed bathymetry is preferred over approximations to better represent hydraulic conditions upstream and a more flexible calibration, estimates can be used to provide reasonable storage for rating curve or structure operation and model stability when data are unavailable and an outlet structure is required. Adding bathymetry improves model stability interaction with the outlet control structure and requires initial conditions when added. Bathymetric approximations can be validated by comparing elevation-storage relationships with available data (e.g., National Inventory of Dams, stream gage or as-builts).

Figure 8 depicts an example of a reservoir where a constant slope bathymetry was approximated based on a storage-elevation curve for a given detention pond to align with the culvert invert. If a reservoir or detention pond outlet is modeled as a culvert structure within HEC-RAS, the bathymetric surface must align with the outlet structure invert.

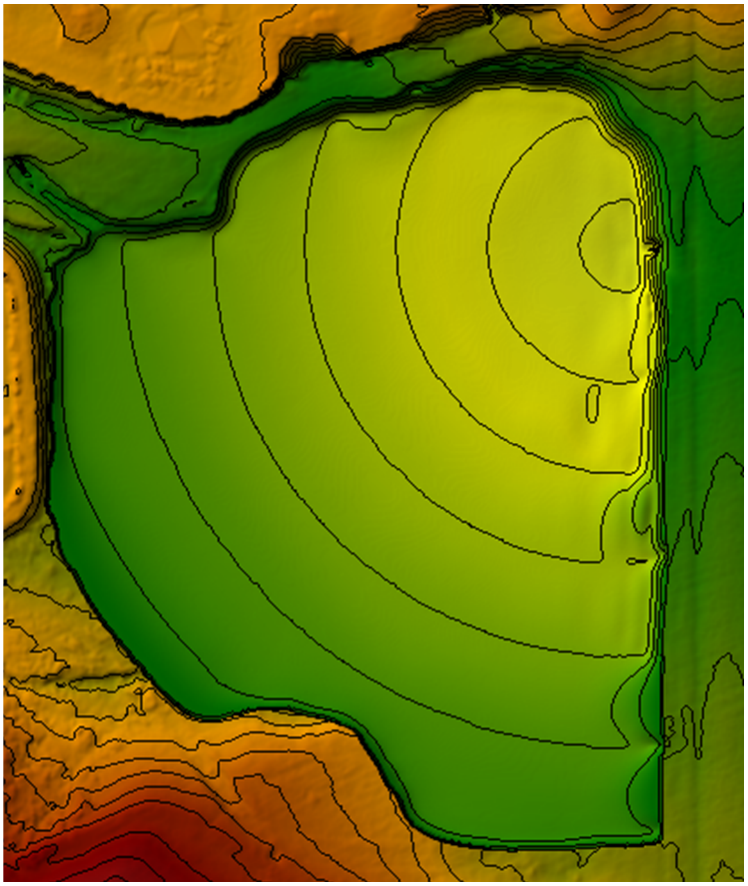


Figure 8. Approximated Reservoir Bathymetry Aligned to Outlet Invert

Where an outlet structure is modeled as an SA/2D connection with an applied rating curve, it is not essential for bathymetry to align with the invert location. In this case, bathymetry can be approximated from the centroid of the hydroflattened surface and rating curves can be established to pull from the center cells where the greatest volume will be available to ensure stability as seen in Figure 9. The bathymetry must be “deep enough” to fall below the lowest elevation on the operational rating curve while still providing sufficient volume to stably transfer via the rating curve without going dry. Rating curves are instantaneous, so the length of the centerline does not impact results for these reservoirs. Elevation-storage curve and elevation-area curve comparisons must be performed where data are available in all cases of approximated bathymetry to ensure reasonable agreement in relationships.

Figure 10 contains an example comparison of the elevation-storage comparison done between as-built published information and the generated terrain surface for the reservoir shown in Figure 9. Where no elevation-storage or elevation-area information is available assume a reasonable slope consistent with the LiDAR surface above the hydroflattened area with the lowest point being either the outlet structure or the storage facility center.

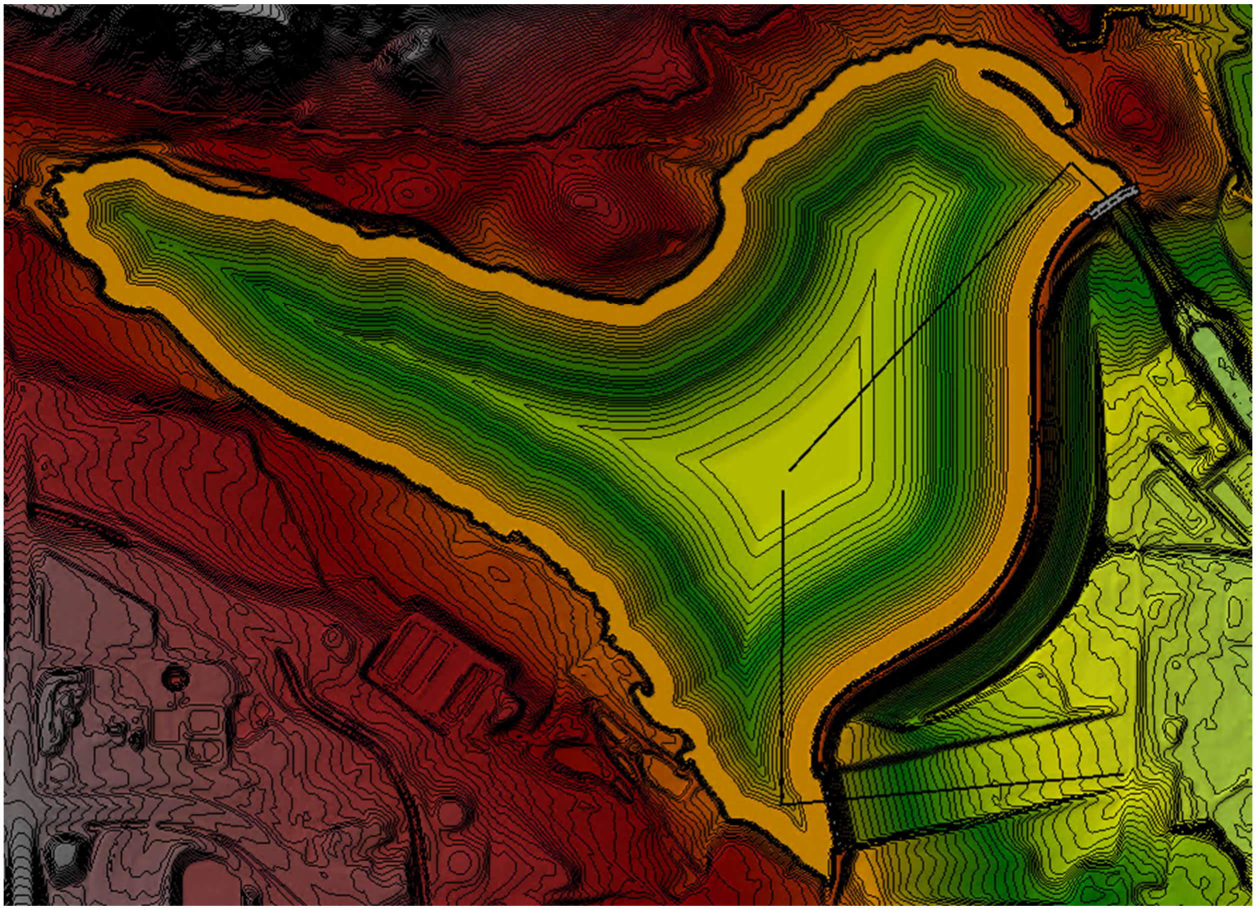


Figure 9. Blunn Reservoir with Rating Curve Outlet Centroid Approximated Bathymetry

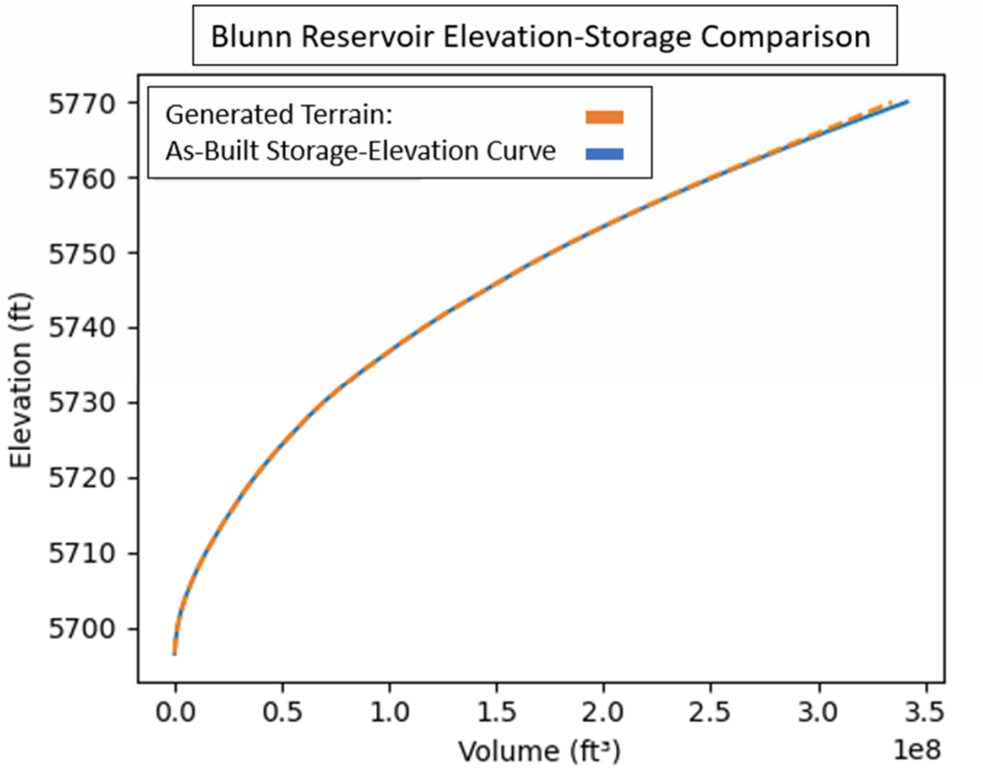


Figure 10. Approximated Bathymetry Elevation-Storage Comparison Curve

4.5 Buildings

To avoid depth rendering artifacts (cupping) and associated distortion of building-level flood risk metrics, building footprints will be represented with only a minor lift of 1.5 feet in raised terrain and then assigned elevated Manning's roughness values as mentioned in Section 5.1.2. Building/roof prints can be obtained from the MHFD receptors dataset, which sources DRCOG (presently 2022), and can be cross-checked with local datasets that may be more current. This minor lift allows for redirection of shallow sheet flow without introducing cupping effects, and then is overtopped by more substantial flows and then subject to high roughness. This rendering artifact issue is well known and may be resolved in future HEC-RAS releases, upon which raising terrain by several feet to fully obstruct flow may be an appropriate method of representing buildings.

This recommendation of a minor lift avoids the rendering artifact observed when building footprints are raised to typical building heights. A thin residual layer of water occasionally registers during a timestep on the sloped roof surface of an elevated building footprint. Although hydraulically insignificant, this isolated sliver of shallow water is treated as a valid wetted cell at the roof elevation during the maximum rendering. As shown in Figure 11, the depth-weighted interpolation then vertically propagates this elevated water surface down the building façade, visually connecting roof-level ponding to surrounding ground-level inundation. This produces bogus vertical extensions of flood depths along the building sides that do not reflect the underlying hydraulic solution. The artifact is evident both in the plan view depth rendering, where water appears trapped atop the structure, and in the extracted profile plot, where the rendered water surface is incorrectly extended to connect with that isolated roof-level water.

These rendering effects inflate localized depth statistics adjacent to the building footprint, directly affecting depth-based metrics used for building-level flood risk assessments. As a result, some individual building risk scores could be overstated due to rendering behavior rather than true hydraulic conditions.

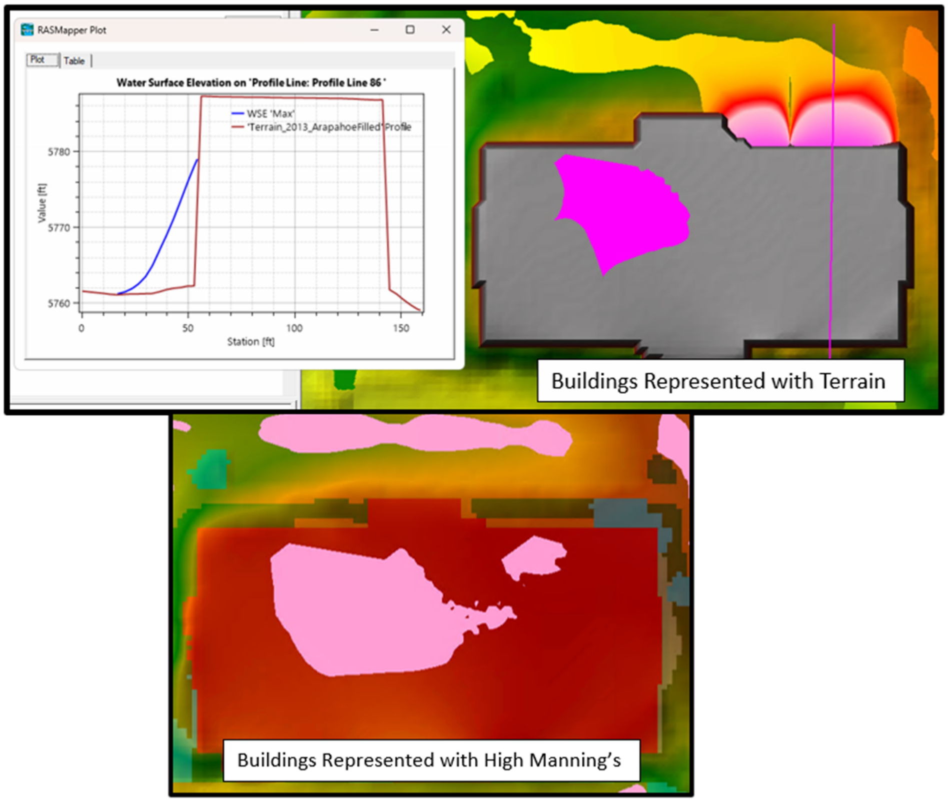


Figure 11. Cupping Artifact with Raised Buildings versus High Manning's Approach

4.6 Terrain Management and HEC-RAS Input

The final model should only contain a single final composite terrain surface. All terrain sources used to construct the final terrain utilized in the model may be included in the supplemental folder submitted alongside the model but should not be linked or included within model folders. All terrain data sources must be clearly documented in the metadata inventory including the use cases and geographic area of coverage. Terrain modifications added to the final terrain layer do not have to be burned into the final geo tiff raster and can remain in HEC-RAS only.

5 Input Layers

Input layers such as land cover, soils, and infiltration parameters are vital for capturing the spatial variability that drives watershed response. For information on topography refer to Section 4. This chapter outlines best practices for generating and applying these layers, including the use of high-resolution datasets and depth-adjusted roughness values. Consistency in input layer development helps hydraulic models to more accurately reflect current conditions and respond appropriately to rainfall events.

5.1 Land Cover Layers

The following section outlines guidance for generating land cover layers to be used on MHFD Flood Risk studies. This section covers domain-wide land cover layers, depth adjustments, and channel Manning's n recommendations. Current FEMA specifications outlined in the "Guidance for Flood Risk Analysis and Mapping," dated December 2020, require land cover to be used to inform roughness coefficients, but do not provide specific requirements on sources and quality.

5.1.1 Base Land Cover Layer

MHFD studies shall use the most recent land cover data from DRCOG, which is presently the 2020 version: DRCOG Land Use/Land Cover (LULC). It is also recommended to check with local governments whether more recent and suitable local datasets exist, particularly for areas experiencing rapid development. Note that the date of the land cover layer should be as close as possible to the terrain date to avoid discrepancies between data sets. More recent data sets should be used with caution as this can generate significant additional level of effort to incorporate and discrepancies between data sets. The DRCOG land cover can be accessed via two primary ways: through the raster and vector download portal for ready-use products, or via the Microsoft Planetary Computer for cloud-optimized analysis and remote sensing workflows.

The model land cover should at minimum contain the following layers obtained from the most relevant DRCOG dataset unless they are absent from the desired study area:

1. Barren Rock
2. Cropland
3. Grassland/Prairie
4. Impervious Surfaces
5. Irrigated Lands/Turf
6. Scrubland/Shrubland
7. Structures
8. Tree Canopy
9. Water

Figure 12 demonstrates an example area in the Goldsmith Gulch watershed DRCOG landcover coverage.

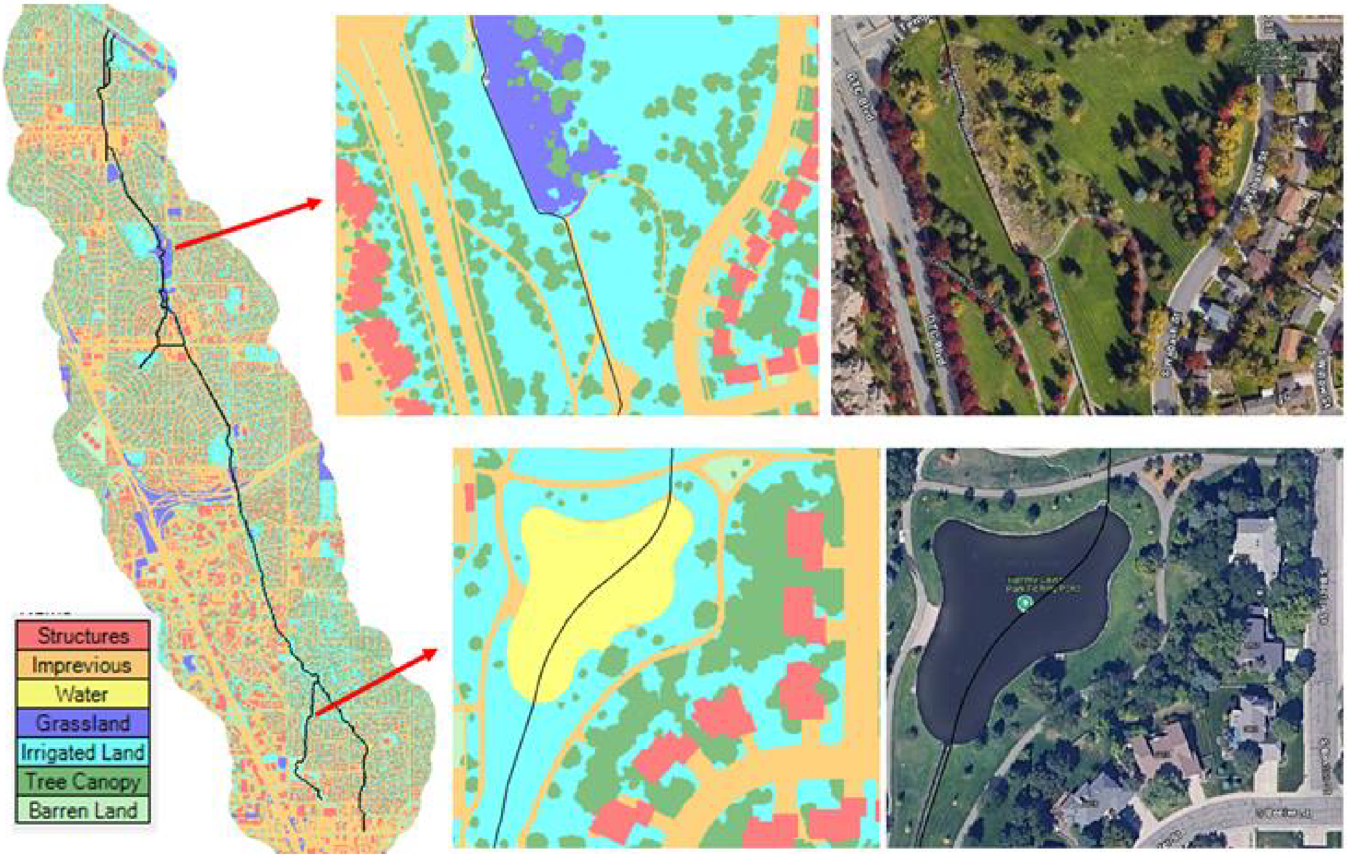


Figure 12. High-Resolution DRCOG Land Cover Layer for the Goldsmith Gulch Watershed

5.1.2 Buildings

Building footprints are already reflected in the DRCOG LULC layer as "structures"; however, depending on the vintage of the LULC collection (presently 2020 is the most recent), there may be more recent building footprints. For watersheds that have been fully built out for some time, no modification of building footprints is merited. For rapidly developing watersheds (e.g., Adams County and Aurora), the most recent building/roof prints should be intersected with the LULC to burn in the building layers on top of the DRCOG LULC. As previously mentioned, buildings will be slightly raised and assigned an elevated Manning's n value of 1.0, ensuring that the spatially varied Manning's "n" option is selected.

5.1.3 Channel Land Cover Region

Channels often have smoother surfaces (e.g., gravel beds, engineered banks) compared to vegetated floodplains, especially when flood depth and velocity are considered. Using low-resolution roughness values typically ignores these differences, causing unrealistic energy losses. Channel roughness directly affects velocity, energy slope, and stage-discharge relationships. If channel roughness is oversimplified or lumped in with floodplain values, the model may misrepresent conveyance capacity. Channel roughness values should be added to the base land cover dataset for all modeled streamlines via calibration regions within HEC-RAS or burned into the land cover layer. The width of the channel-specific roughness value features should be based on the top of bank or edge of water where it is visible in LiDAR or aerial imagery. Channel Manning's "n" values may change based on channel characteristics throughout the model and will likely not be uniform throughout a study area.

5.1.4 Depth-Adjusted Land Cover

While it is broadly accepted that Manning's "n" values are depth-dependent, there is little literature documenting methodologies or factors by which to adjust the values as final values are largely established during calibration. An important component of this is shallow-overland flow, as represented by depth-weighted roughness values being notably higher than typical channel flow. Therefore, values will be adjusted as part of calibration to affect hydrograph shape and timing, while being kept within a reasonable range based on engineering judgment.

Examples of depth-weighted Manning's "n" values for the most common DRCOG landcover classes can be found in Table 4. *Please note that these values are provided only as example starting points and may be used as initial inputs for model setup. They should be reviewed and adjusted during calibration as needed to achieve acceptable performance based on the specific conditions and calibration metrics of each study area.* The spatial bands of the depth-weighed Manning's "n" layers do not have to be iteratively developed, but their values should be varied throughout calibration efforts. Once preliminary geometries are developed, depth-weighted layers should be generated based on the 100-year event and events of smaller magnitude if necessary for calibration purposes.

Table 4. Potential Starting Values for Depth-Weighted Manning's "n" (example only)

Land Cover	Sheet Flow (<0.25 ft)	Shallow Flow (0.25-1 ft)	Normal Flow (>1 ft)
Grass	0.12	0.08	0.04
Turf	0.08	0.06	0.035
Cropland	0.13	0.085	0.045
Trees	0.10	0.10	0.10
Buildings	1.0	1.0	1.0
Open/Barren Land	0.10	0.07	0.03
Water	0.03	0.03	0.03
Concrete	0.040 - 0.06	0.03	0.025
Asphalt	0.035 - 0.05	0.02	0.016

5.2 Soils

Soils data can be sourced from SSURGO and must cover the entire model domain. Review the dataset carefully to identify and fill any gaps with the most conservative soil group or texture in its immediate neighborhood. Also check for inconsistencies in soil classifications along political boundaries and resolve them using available data or alternative sources.

6 Hydrology and Boundary Conditions (Pluvial)

In the FRA 2D RoM models, hydrology takes the form of precipitation. Precipitation refers to applied precipitation as losses will be determined by the infiltration layer of the geometry. Externally applied inflow hydrographs will also be utilized to link basins where the inflow of one hydraulic model flows into a downstream study area. The routing of overland runoff comprises unsteady flows throughout the mesh and eventually concentrates into established flow paths that become creeks and rivers.

This hydrology chapter addresses the selection and application of design storms, infiltrations and losses, calibration and validation methods, regulation and reservoir representation, inflow management, corridor modeling, and boundary conditions. Consistent and standardized hydrologic inputs are essential for defensible flood risk assessments to arrive at model results that are reasonable, transparent, and reproducible.

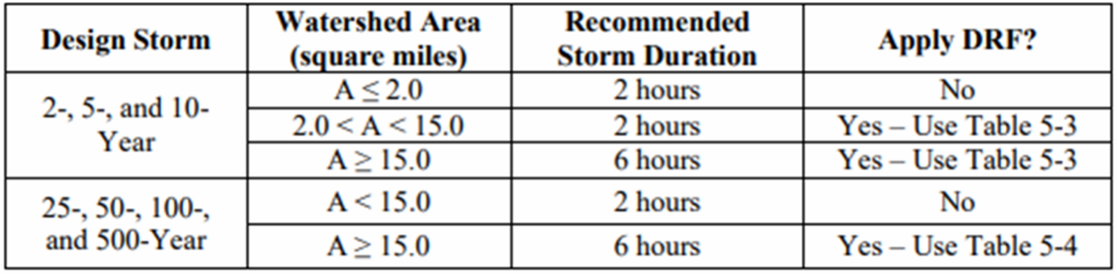
6.1 Design Storms

In the context of the MHFD Flood Risk Program, the term “design storms” refers to synthetic events used for predictive analysis following a prescribed duration and distribution and are deterministically-assigned values based on recurrence interval. For the purposes of this Program, design storms are not intended for ready-application in stream design projects, instead the term is meant to convey a recurrence-based synthetic event.

The recommended source for precipitation is the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 annual maximum time series (AMS) data, which is provided for 10 recurrence intervals from 1-year to 1000-year and 19 storm durations from 5 minutes to 60 days. If a recurrence interval required for flood risk analysis is not listed on NOAA Atlas’s database (e.g., the 400-year event), then accumulated precipitation totals should be interpolated between the listed recurrence interval values. NOAA Atlas 14 accumulated precipitation totals must consider the spatial variability of the watershed depending on its size and physiogeographic setting, such that a single value from its centroid might suffice for a small watershed but spatially varied precipitation will be required for larger watersheds. The process and guidance for developing spatially varied precipitation layers with depth area reduction factors applied is outlined in Section 6.1.2.

The design storm distribution and design is determined via the Colorado Urban Hydrograph Procedure (CUHP), where the duration will be either 2 or 6 hours depending on the recurrence interval and watershed area, as determined by Table 5-1 found in chapter 5 of MHFD’s Urban Drainage and Flood Control District Urban Storm Drainage Criteria Manual, included here as Table 5.

Table 5. Storm Duration and Area Adjustment (Table 5-1 from USDCM)



Design Storm	Watershed Area (square miles)	Recommended Storm Duration	Apply DRF?
2-, 5-, and 10-Year	A ≤ 2.0	2 hours	No
	2.0 < A < 15.0	2 hours	Yes – Use Table 5-3
	A ≥ 15.0	6 hours	Yes – Use Table 5-3
25-, 50-, 100-, and 500-Year	A < 15.0	2 hours	No
	A ≥ 15.0	6 hours	Yes – Use Table 5-4

*Watershed area is defined by the largest individual watershed within the model domain, which will be smaller than the model domain when adjacent watersheds are present in a single geometry.

6.1.1 Required Design Storm Recurrence Intervals

The following 13 design storms are used in the MHFD flood risk assessment process. As previously mentioned, these are not meant for design applications, but the term conveys a predictive synthetic event tied to a deterministic recurrence interval. It is worth noting that frequent, smaller events are included to fill the probability space, as they carry the most weight when aggregated or annualized.

1-Year
2-Year
5-Year
10-Year
25-Year
50-Year
100-Year
200-Year
300-Year [1]
400-Year [1]
500-Year
750-Year [1,2]
1000-Year

[1] Interpolated log-log between values reported by NOAA Atlas 14, which involves linearly interpolating between the log of the recurrence interval (x-axis) and the log of the precipitation depth (y-axis).

[2] The 750-Year is now a required design event for Class 3 structures (e.g., large assembly buildings like theaters, schools, detention facilities, nursing homes, etc.) per ASCE/SEI 24-24 Building Code (Flood Resistant Design and Construction).

6.1.2 Depth Area Reduced Gridded Rainfall

This section describes the derivation of spatially varied gridded rainfall for design storms with depth-area-reduction factors (DARFs) applied for use in 2D HEC-RAS modeling.

Developing the gridded rainfall follows the rainfall distribution framework in Chapter 5 of the Mile High Flood District USDCM, which provides tabulated DARFs for CUHP design storm development by return period, storm duration, and watershed area in Tables 5-3 and 5-4. Because point precipitation estimates generally exceed watershed-average rainfall depths, DARF adjustment may be applied when extending point rainfall to larger drainage areas. The discussion below first summarizes the standard CUHP storm-development procedure (termed SCALE) and then describes the modified tail-trim implementation (termed TRIM) used for this gridded rainfall application. For reasons described below, the TRIM method is recommended for the Flood Risk Program. An illustrative comparison (Figure 13) is provided below between the two methods for both incremental and cumulative rainfall depths at a test location within the Ralston model domain.

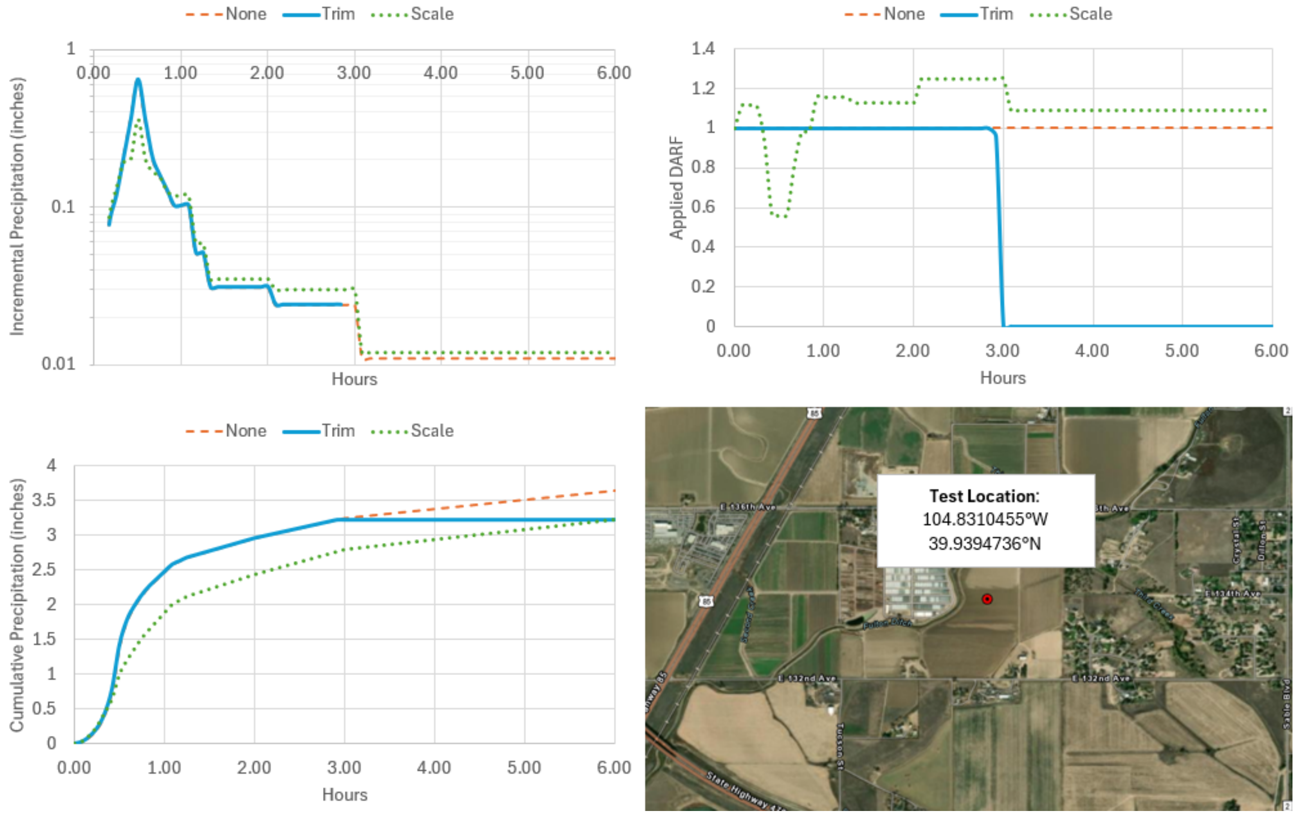


Figure 13. Example Hyetograph Comparison between DARF Methods

The design storm rainfall grids should be generated as either 2 or 6-hour hyetographs in DSS format from NOAA Atlas 14 precipitation-frequency rasters following the CUHP methodology in USDCM Chapter 5, Sections 3.1 and 3.2. Doing this in a gridded fashion (rather than at a single location) involves clipping the 1-hour, 3-hour, and 6-hour NOAA Atlas 14 precipitation rasters to the model domain, constructing a 5-minute incremental hyetograph, and applying DARFs where watershed area and return period warrant adjustment. The 2 and 6-hour storm grids should be assembled following the standard CUHP distribution. Using the 6-hour duration as an example, the first two hours follow Table 5-2, the third hour is based on the difference between the 3-hour and computed 2-hour depths, and hours four through six are based on the difference between the 6-hour and computed 3-hour depths. The temporal distribution is constructed at 5-minute resolution over 6 hours, yielding 72 rainfall increments.

SCALE (Typical Approach for 1D)

This section describes how the typical approach used in 1D could be applied directly to gridded rainfall; however, this is not the recommended approach and is included only for comparison purposes. For each grid cell, the first 24 increments, covering minutes 5 through 120, should be computed by multiplying the NOAA Atlas 14 1-hour precipitation depth by the CUHP Table 5-2 temporal percentages for the selected return period. CUHP describes this distribution as a calibrated design storm for distributed rainfall-runoff routing models and notes that the 2-hour distribution totals approximately 115.6 to 115.7 percent of the 1-hour point rainfall depth, depending on return period. This logic is implemented using embedded Table 5-2 fractional distributions for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-year events, with interpolation for non-tabulated return periods.

The third hour of the storm is computed as the residual difference between the 3-hour precipitation depth and the accumulated 2-hour CUHP distribution, with that residual distributed uniformly across the twelve 5-minute increments from minutes 125 through 180. The fourth through sixth hours are then computed as the residual difference between the 6-hour precipitation depth and the accumulated 3-hour storm depth, distributed uniformly across the thirty-six 5-minute increments from minutes 185 through 360. This follows CUHP guidance for constructing design storms.

For the standard CUHP DARF application, the design storm increments should be modified using time-step-specific correction factors from the appropriate CUHP table. For the 2-, 5-, and 10-year events, the minor-event table corresponding to USDCM Table 5-3, with DARF applicability beginning at a watershed area of 2-square miles is used. For the 25-year and larger events, the infrequent-event table corresponding to USDCM Table 5-4, with DRF applicability beginning at 15 square miles is used. In its direct scaling form, the DARF adjustment can be represented as

$$P'_{t,x,y} = F_t(A,T)\, P_{t,x,y}$$

Where $P_{t,x,y}$ is the unadjusted incremental rainfall depth at time step (t) and raster cell (x, y) $F_t(A,T)$ is the CUHP depth-area correction factor for watershed area (A) and return period (T) and $P'_{t,x,y}$ is the DRF-adjusted incremental rainfall depth. In the direct-scaling implementation, each of the 72 time-step grids should be multiplied by the corresponding DARF factor from the appropriate CUHP table, using interpolated area-specific values where needed.

TRIM (Proposed Approach for RoM)

For 2D RoM modeling, a modified tail-trim method should be used instead of reducing depth at each time-step. The objective of TRIM is to reduce the total storm-volume by the CUHP DRF factors without attenuating the short-duration rainfall intensity at the peak that drives local runoff generation and overland flow response in a rain-on-grid 2D hydraulic model. Rather than reducing rainfall throughout the storm, the tail-trim method uses the conventionally DARF-adjusted hyetograph to define the target total depth for each grid cell and removes the equivalent net volume from the low-intensity trailing portion of the 6-hour storm.

The resulting rainfall boundary condition supplied to HEC-RAS should be a spatially distributed 2 or 6-hour duration hyetograph in which each grid cell retains its NOAA Atlas 14-derived spatial depth pattern, where the CUHP distribution is preserved, peak intensities unaltered, and the total rainfall volume is adjusted consistently with the applicable CUHP DRF factors. This approach is well suited to RoM simulations because it preserves the spatially variable precipitation field rather than replacing it with a lumped basin-average rainfall series, while still accounting for the recognized reduction in areal-average rainfall depth over larger watershed domains. In this way, the workflow provides a reproducible bridge between the CUHP design-storm methodology and modern 2D hydraulic modeling practice.

Recommendation

There are fundamental differences between RoM hydrology and traditional FHAD hydrology via CUHP/SWMM. RoM applies rainfall to every cell throughout the entire grid covering the watershed, meaning that every drainage of any size is being modeled in the same shared simulation. Whereas

SWMM can run multiple simulations with varied DARFs and extract flows only corresponding to the select basins, RoM has hydraulics coupled with hydrology and sources the shared simulation (one cannot select flows across multiple DARF scenarios). Therefore, the TRIM method is appealing as it reduces volume without reducing intensity. Typically flooding is driven by total volume for the larger streams (near the domain outlet) but driven by intensity for the smaller drainages in the headwaters. Reducing volume while maintaining intensity has a similar effect to applying DARFs in mainstem but not in the headwaters. This approach does so in gridded data to account for spatial variability of the precipitation depths across the watershed, which can be fairly variable in the District moving from west to east due to the significant changes presented by the foothills to plains transition.

6.1.3 Conservative Assumptions and Restart File

RoM models used for MHFD Flood Risk must use a restart file (also often called a hot start file) to consume micro-storage in the watershed prior to applying design storms. This application complies with the District's long-held policy of only accounting for planned storage through built infrastructure. This is an appropriate practice for future planning applications, including regulatory and managing development; however, careful consideration must be given to whether restart files ought to be used (or their resulting flows referenced) for stream design projects as a restart file brings an intentional degree of conservatism that is variable and can be substantial.

The use of a restart file introduces initial conditions where the micro-storage is consumed (pre-filled) prior to a storm event. A restart file is generated by applying a large storm event (chosen as the 50-year design storm following the CUHP) to the final geometry and allowing all excess runoff to fully drain out of the model, leaving ponded water behind throughout the watershed in natural and man-made depressions. Most depressions without an outlet, hydro-connectors, or structure will fill and lose their attenuating effect once the design storm is applied.

The net impact of the restart file will increase runoff volume and increase flashier peak flows when used with the design storms, and the magnitude of impact is watershed specific. Flatter topography with more undulations tends to hold back a significant volume of overland water resulting in a considerable difference in runoff and flow when applied. Watersheds with topographic relief that is well-drained will retain less overland water and result in smaller increases in flow and volume.

It is worth noting that using a restart file only changes the initial hydraulic conditions, but not the initial hydrologic conditions for the simulation. This is different for HEC-HMS where a restart file would capture hydrologic conditions (including soil moisture, abstraction, etc.); however, HEC RAS operates differently and captures only the hydraulic conditions in the restart file and completely resets the hydrologic conditions for the simulation. Therefore, infiltration will still occur in every single cell for HEC-RAS RoM regardless of whether the cell is dry or underwater, as the losses are actually removed from the applied hyetograph before ever "touching" the hydraulic surface. Once rainfall has losses removed from the applied hyetograph, the excess precipitation hyetograph is converted to permanent runoff, which can subsequently leave the model only through boundary conditions, pipes, or pumps – but infiltration ceases to occur.

The restart file should be created from the fully drained conditions at the end of 6 days or more after applying a 50-year design storm event. It is recommended that the 50-year restart plan be run for at least 6 days. However, for smaller reaches where full drainage is obtained sooner, shorter simulation durations are accepted. Initial conditions are represented for all 13 design storms through the use of one shared restart file, but this is not used for calibration or validation events. Figure 14 illustrates example initial conditions introduced via a restart file plan. Figure 15 is an example of a 50-year restart file plan run used to generate a restart file for all design storms.

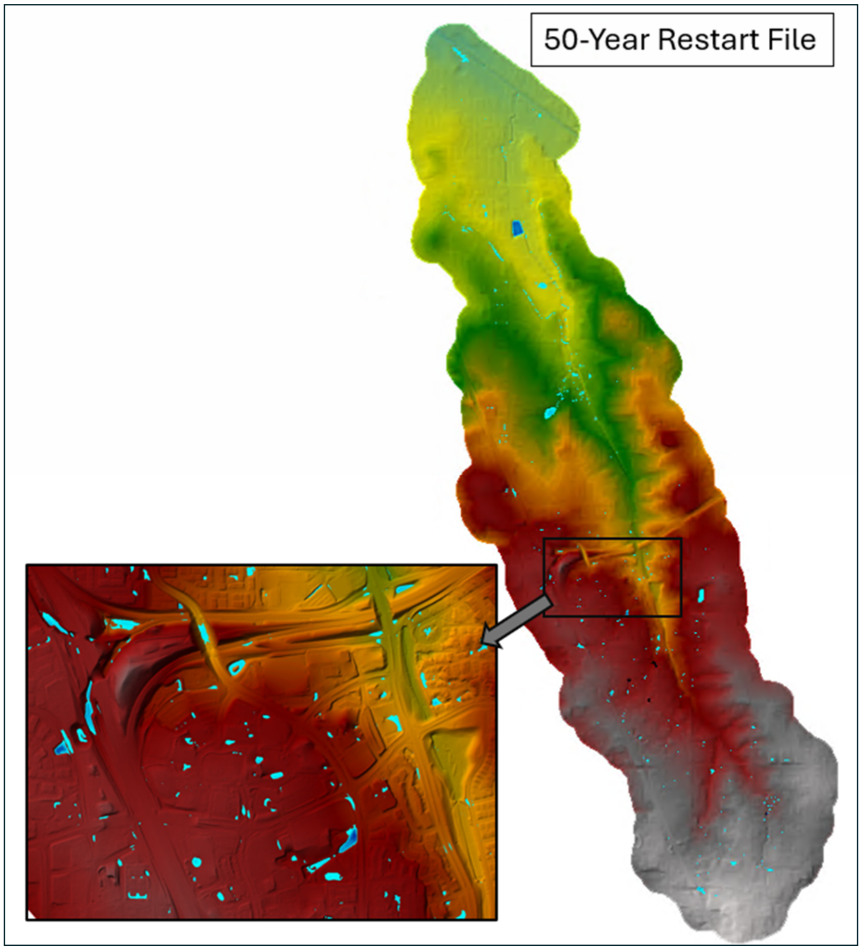
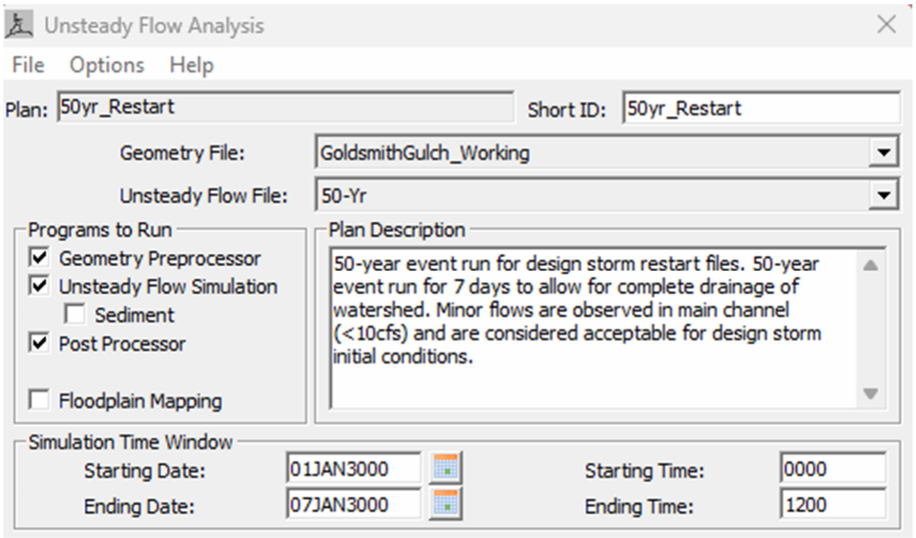


Figure 14. Goldsmith Gulch Restart File at 7.5 Days



Plan: 50yr_Restart
Short ID: 50yr_Restart
Geometry File: GoldsmithGulch_Working
Unsteady Flow File: 50-Yr

Programs to Run
☑ Geometry Preprocessor
☑ Unsteady Flow Simulation
☐ Sediment
☑ Post Processor
☐ Floodplain Mapping

Plan Description
50-year event run for design storm restart files. 50-year event run for 7 days to allow for complete drainage of watershed. Minor flows are observed in main channel (<10cfs) and are considered acceptable for design storm initial conditions.

Simulation Time Window
Starting Date: 01JAN3000
Ending Date: 07JAN3000
Starting Time: 0000
Ending Time: 1200

Figure 15. Restart File Plan Settings

6.2 Infiltration

The SCS Curve Number Soil Conservation Service CN method is the default infiltration method for 2D RoM modeling of design storms in the District. For short, single peak design storms and calibration events, all infiltration methods perform similarly; however, the relative simplicity and familiarity of the

CN method make it easiest to standardize and implement in the District. Since the current MHFD framework results in design storms of 2 or 6 hours, using CNs is preferred as the advantages of Green-Ampt (G&A) with variable soil moisture and infiltration are not beneficial over such short durations.

Outside of design storms for Flood Risk Assessment, the Green-Ampt (G&A) method may be preferred for longer continuous rainfall events (such as select historic events like the 2013 event) due to its ability to redistribute soil moisture during breaks in precipitation. This partial recovery of infiltration capacity between heavier rainfall leads to more realistic modeling of runoff, producing lower peak flows that closely align with observed hydrographs. Although the calibration may be more complex, G&A enables a notably improved ability to reflect longer duration and multi-peaked historic events. Although uncommon, if the G&A method is the only suitable approach to calibrate an extended storm, it should also be used for the design storms and would also need to be calibrated against a shorter duration, single peak event (if possible). However, for the single-peak, short duration design storms used for Flood Risk, the CN method is the default approach.

Regardless of the method used, infiltration parameter values and sources must be clearly documented in the summary report and model reference folder, particularly when justifying assumptions or deviations from literature values. An example table with common DRCOG land cover classes and initial Curve Numbers is provided in Table 6, with values sourced from the TR-55 tables. These values should be treated as starting points and ought to be adjusted during calibration to better reflect site-specific conditions. This table is not exhaustive, as additional land cover classes may be present in other DRCOG study areas, yet is intended to provide a framework for mapping local land cover classes to appropriate NRCS TR-55 categories (NRCS, 1986).

Table 6. Potential Starting Values for Curve Number Assignments

Land Cover : Soil Group	Cure Number	TR-55 Reference
Buildings : NoData	98	
Buildings : A	98	
Buildings : B	98	
Buildings : C	98	
Buildings : D	98	
Impervious : NoData	98	
Impervious : A	98	
Impervious : B	98	
Impervious : C	98	
Impervious : D	98	
Water : NoData	98	
Water : A	98	
Water : B	98	
Water : C	98	
Water : D	98	
Grassland : NoData	74	Herbaceous Grassland
Grassland : A	49	
Grassland : B	62	
Grassland : C	74	

Land Cover : Soil Group	Cure Number	TR-55 Reference
Grassland : D	85	
Tree : NoData	70	Mixed Forest
Tree : A	30	
Tree : B	55	
Tree : C	70	
Tree : D	77	
Turf : NoData	79	Developed Open Space
Turf : A	49	
Turf : B	69	
Turf : C	79	
Turf : D	84	
Barren Land : NoData	88	Barren Land
Barren Land : A	74	
Barren Land : B	83	
Barren Land : C	88	
Barren Land : D	90	
Cropland : NoData	78	Cultivated Crops
Cropland : A	62	
Cropland : B	71	
Cropland : C	78	
Cropland : D	81	

6.3 Regulation and Reservoirs

Regulated reservoirs are required to be modeled in accordance with their documented operational discharge and design curves. It is standard practice to assume that all reservoirs are at their normal pool elevation at the onset of a simulation for design events. The initial pool elevation may vary for the historical events used during calibration depending on available information or gage data. The starting elevation can be established through initial condition settings, base flow assignments, or by modifying the terrain within RASMapper to reflect the normal pool level. Modifying the terrain elevation to reflect normal pool level may lead to instabilities at the outlet structure depending on the volume capacity of the inlet cell. If necessary, storage may be created at the outlet to improve model stability.

When representing regulated dams within a hydraulic model, it is recommended that the reservoir outlet or spillway be modeled using a rating curve that accurately reflects the relationship between the upstream WSE and outflow discharge, as defined by the dam’s operational rules and physical characteristics. A good starting point is to reference existing SWMM models developed for FHADs, which can help identify which ponds should be included and provide readily available rating curves for use in the model. The rating curve covers the full spectrum of flows and extends to a WSE exceeding the dam crest such that the simulated WSE will not exceed the rating curve, as that would throw a warning in the model.

If additional public detention facilities are identified without available information from existing SWMM models or as-built data, they should be evaluated in coordination with the District. In general, inclusion should be based on the facility’s influence on the modeled drainageways and infrastructure, along with how its operational timing aligns with the terrain and land cover conditions used in the study. These decisions should be clearly documented for each facility in the accompanying report or technical memo, including the basis for inclusion or exclusion, data sources used, assumptions made, and any limitations, so the approach is transparent and can be revisited in future updates.

Private dams and ponds should *not* be modeled as providing flood storage, detention, or attenuation unless there is written adequate assurance for provided storage. Under typical conditions, these dams must be prefilled so that inflow equals outflow. This can be achieved by applying an appropriate initial WSE or by modifying the terrain to reflect a pre-filled water surface. In other circumstances it may be acceptable to represent these dams with intentionally leaky cells over the dam crest or through a terrain modification to flatten the crest. With this approach, any runoff entering the dam simply spills over the spillway crest without altering flow timing or reducing peak discharge.

6.4 Inflows and Basin Linking

To maintain hydraulic consistency across linked model domains, outflows from an upstream model are applied as inflows to the downstream model. When these external inflows are combined with locally applied precipitation, minor adjustments to inflow timing or magnitude may be required to maintain reasonable interaction with downstream hydraulics which are detailed below.

When the outlet of the upstream domain is near a gage used during its calibration, hydrographs generated for that outlet can generally be incorporated into the downstream model with little to no modifications to timing and peak flows for instances where the upstream model was well calibrated. If the gage is significantly upstream of the outlet, the upstream domain was not calibrated to a gage, or if the upstream domain is unstudied, any imported or newly generated design inflows must be verified against a gage present in the downstream domain, if possible. Any modifications to the inflow for synthetic events should be dictated by the downstream gage calibration (Section 10), preferably after watershed response parameters such as CN and Manning's roughness have been calibrated beforehand to historical event(s).

When adjustments to inflow hydrographs are necessary, the overall hydrograph shape should remain unchanged, with modifications limited to scaling the magnitude of flow values and applying minor timing shifts to better align inflow generated peak flow with downstream rainfall-runoff response. Timing adjustments should only be used to shift the hydrograph as a whole, not to distort its shape. If additional refinement is required, or if the gage is present outside of the limits of the downstream domain, a gage projection analysis may be performed to supplement the adjustment of inflow hydrographs (see USDA - Appendix D, Section D.2.2.1 for guidance). In systems where coincident peak timing between upstream inflows and locally generated runoff is uncertain or unlikely, joint probability methods may be required to define appropriate inflow combinations, as described in Section 6.5.

Additionally, if the flow exchange boundary is near a flow-regulating dam, it is likely advantageous to consider resetting flows that occur due to storage and discharge regulation by the dam. In such cases, the upstream and downstream domains should overlap, with the upstream domain extending slightly downstream of the dam crest or outlet. Regulated outflows from the dam should then be inserted as inflows into the downstream domain.

6.5 Corridor Modeling and Joint Probability Analysis

For larger watersheds with separate and linked domains as inflows, it is appropriate to use Joint-Probability Analysis (JPA) to define hydrologic inputs at confluences where the timing of tributary and mainstem peaks is typically not synchronized. In addition to storm movement, different watershed characteristics and configuration contribute to different times of concentration suggesting that coincident peaks with matching recurrence intervals between a mainstem and tributary is highly unlikely. JPA aims to capture a similar reduction effect as DARFs that recognize it doesn't rain the same everywhere at once, and this too recognizes that rainfall is variable and moves across a domain such that peaks are typically offset (not coincident) and of varying magnitudes (different recurrence intervals). Whether JPA should be applied is typically governed more by the drainage area ratio (DAR) than by the size of the main basin area. For the purposes of MHFD Flood Risk, a basin size threshold of 15 sq. mi. is proposed for whether to use JPA. If there is significant upstream regulation that considerably affects timing and flows, the main basin size should consider the unregulated portion downstream of the reservoir or feature.

When developing 2D surface hydraulic models for larger rivers with tributaries, a key decision is how to represent tributary inflows at confluences the two primary options are to:

- Use actual hydrographs from the tributaries (e.g., from hydrologic models or observed data).
- Apply JPA to define statistically likely combinations of tributary and mainstem flows.

Research and national guidance provide a strong foundation for evaluating JPA. The National Cooperative Highway Research Program (NCHRP) Report 15-36 (Kilgore et al., 2013) introduced practical JPA procedures, recommending a copula-based method (specifically the Gumbel–Hougaard copula for its flexibility) and an alternative total probability method. These methods were developed using a national dataset of gaged confluences and are designed to identify the most probable and hydraulically significant combinations of flows for design purposes.

Building on this, the Federal Highway Administration's Hydraulic Engineering Circular No. 19 (HEC-19, 2023) integrates JPA into highway hydrology and provides a step-by-step procedure for confluence analysis. HEC-19 emphasizes that if tributary and mainstem flows show any significant correlation or interdependence, a joint-frequency analysis should be performed. Conversely, if peaks are shown to be uncorrelated, tributary and mainstem flows can be treated independently.

The following aspects should guide the modeling approach:

- **Drainage Area Ratio (DAR):** If the tributary's drainage area is less than 30% of the mainstem's (DAR <0.3), tributary and mainstem peaks are unlikely to coincide. In such cases, JPA is preferred, as it avoids overestimating flood risk associated with assuming simultaneous 100-year peaks. Ghanghas et al. (2023) found that DAR is a strong predictor of flow correlation and that using JPA in these cases leads to more efficient and realistic designs.
- **Stream Order and Regulation:** Tributaries with lower stream order or flashy, unregulated hydrology tend to peak earlier than regulated mainstems. Mainstems with large reservoirs or upstream controls often have delayed and attenuated peaks. This temporal mismatch supports the use of JPA, as outlined in NCHRP 15-36. Significant regulation or reservoirs with attested flood storage may support noncoincidence such that the one flooding source passes the peak while the other is near baseflow conditions.
- **Peak Timing and Correlation:** Analyze historic stream gage data to assess peak alignment. If the correlation between tributary and mainstem peaks (e.g., Kendall's Tau) is moderate (0.5–0.8), JPA is appropriate. If correlation is high (>0.8), actual hydrographs should be used to capture coincident flood behavior. If correlation is very low (<0.3), the tributary and mainstem can be treated independently. These values are influenced from the findings of Ghanghas et al.(2023).

When JPA is implemented in a riverine domain, the model boundary and mesh can remain unchanged, because the tributary inflows can be injected directly into the river channel as internal inflows (see Section 6.6.1). It is the District's preference to preserve one mesh and geometry for both RoM and PSS models (Section 11) for long-term maintenance and versioning control, rather than creating a trimmed domain in a separate, maintained geometry and thereby introducing multiple versions.

6.6 Boundary Conditions

Boundary conditions define how water enters and exits the model domain, influencing flow continuity, magnitude, and stability. This chapter explains the setup of internal and external boundary conditions for RoM models, including best practices for hydrograph insertion, normal depth assignments, and perimeter buffering. While applied rainfall is an internal boundary condition via a hyetograph (see Section 6.1), this section is focused on inflows and outflows. Consistent boundary condition implementation is key to achieving stable simulations and seamless integration/linking between adjacent models. Boundary conditions for PSS models are handled differently, without applied rainfall and using incremental internal inflows, as discussed in Section 11.

6.6.1 Internal Boundary Conditions

Internal boundary conditions should be used at select locations for one or more of the following applications:

- To convey flow changes, either for riverine-only domains (Section 11) or to capture tributary inflows without trimming the mesh to accommodate an external boundary condition from an upstream model.
- To establish baseflow and/or initial conditions for reservoirs and channels before precipitation is applied.
- To force flow changes (manual overrides) that offset inadvertent storage (e.g., undersized culverts) or to force a riverine-only model to be pseudo-steady state. Neither of these are currently recommended for Flood Risk and are not covered further here.

Internal inflow hydrographs can be inserted at specific locations along the scoped river to model flow change locations along it. For example, on major rivers with large drainage areas and regulated flows (e.g., the South Platte River), these tributary additions should be modeled as non-coincident. In such cases, JPA as outlined in Section 6.5 should also be performed to find the appropriate coincident peak flows at the river-tributary confluences. These flows should also be inserted using an internal boundary condition line just upstream of the confluence on the mainstem.

Baseflows may be added for both design storm events as well as historical events to assist with calibration. Baseflows should be added in streams with strategic placement based on defensible historical data. Common examples of baseflow locations include downstream of reservoirs at the upstream end of watersheds, such as Ralston Reservoir above the Ralston Creek watershed in Arvada or upstream of developed areas in undeveloped spaces where risk assessment will not be impacted. Baseflows should be added where downstream gages show clear indications of consistent channelized flow over extended periods of time. For historical events baseflows should be determined by historical gage records and set to values that will assist in the calibration process. For design storms baseflows should be set to the average baseflow observed over the past 5-10 years at ALERT gages. For ungauged streams, baseflows should be determined based on institutional knowledge (e.g., field observations, familiarity with the stream system) or based on nearby reaches similar in size and land use. The default for intermittent drainages and dry washes should be without baseflows.

While the preferred method is to use the same geometry with an internal boundary condition, another viable approach is to trim the geometry to the riverine corridor and apply external boundary conditions at tributaries. This latter approach slightly reduces file size and may present a more "focused" option for model management. However, it may also introduce additional work and potential versioning concerns where future mesh edits would need to be made in both geometries, and so such an approach is not recommended.

6.6.2 External Boundary Conditions

External boundary conditions must be placed at the watershed boundary at all locations where water enters or leaves the model bounds. External boundary condition types may vary depending on conditions and can be represented by normal depths, stage hydrographs, or flow hydrographs depending on the model conditions. External boundary conditions defined with normal depth should be set to the slope of the channel or flow path leaving the model and must be placed with careful consideration for changes in topography. Where there is notable variability in terrain, especially between the floodplain and overbanks, multiple individual external normal depth boundary conditions should be placed with energy gradient slopes representative of their profile. In general, it is best practice to capture the width of the largest floodplain at the outflow with a single boundary condition (facilitating better model flow and volume accounting), and then to place adjacent and separate boundary conditions along the overland.

If a model boundary is not delineated to precisely match the watershed boundary, it is recommended to add a normal depth external boundary condition surrounding the perimeter to remove unintentional ponding at the model perimeter. Alternatively, a rating curve may be used if hydraulic information is available for the outflow condition such as a dam or reservoir outlet. Where downstream boundaries are major waterways, such as the South Platte River or Clear Creek, it is recommended to include overlap into the major channel with a baseflow applied in order to properly capture hydraulics at the confluence. In this case the external boundary condition will be on the major channel and set to the channel slope for the major channel. Figure 18 demonstrates an example of a major channel confluence boundary condition layout. In flatter terrain or heavily urbanized areas with less defined watershed divides, it is often best practice to intentionally buffer the watershed boundary by several hundred feet (typically 400 to 800 feet) to ensure that all drainage paths to the study area are captured. In such cases, all or nearly all of the outer portions of the model domain will drain away from the main stem and require a surrounding normal depth boundary condition to prevent "glass walls" or retained ponding. Example boundary condition scenarios are shown in Figure 16 and Figure 17.

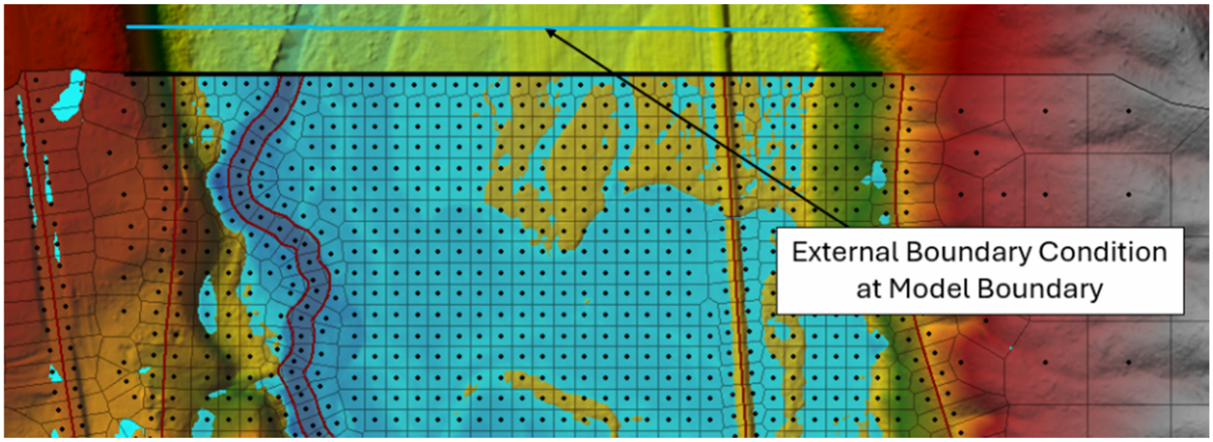


Figure 16. External Boundary Condition on Channel End and Model Boundary

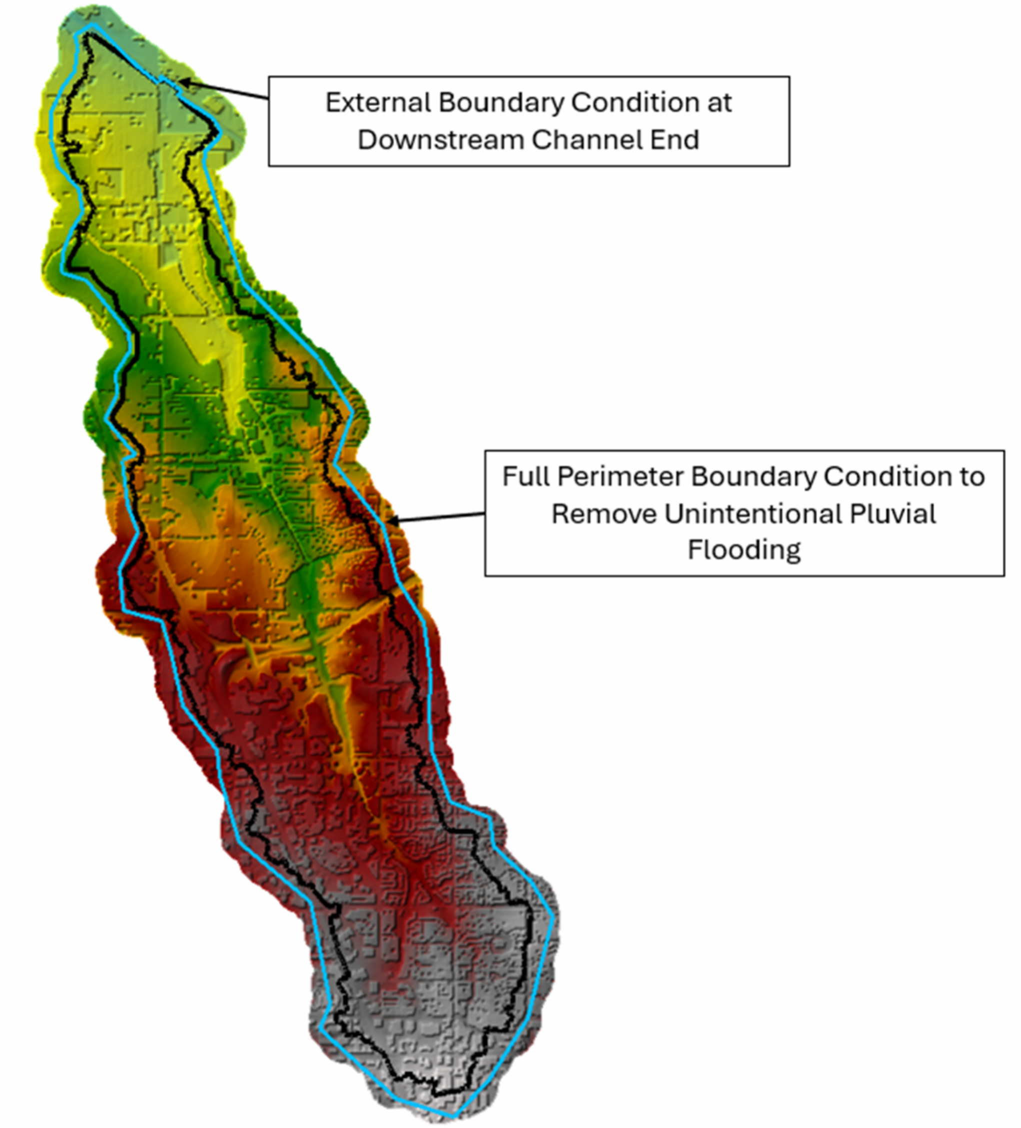


Figure 17. Full Perimeter Boundary Condition to Remove Pluvial Flooding at Perimeter

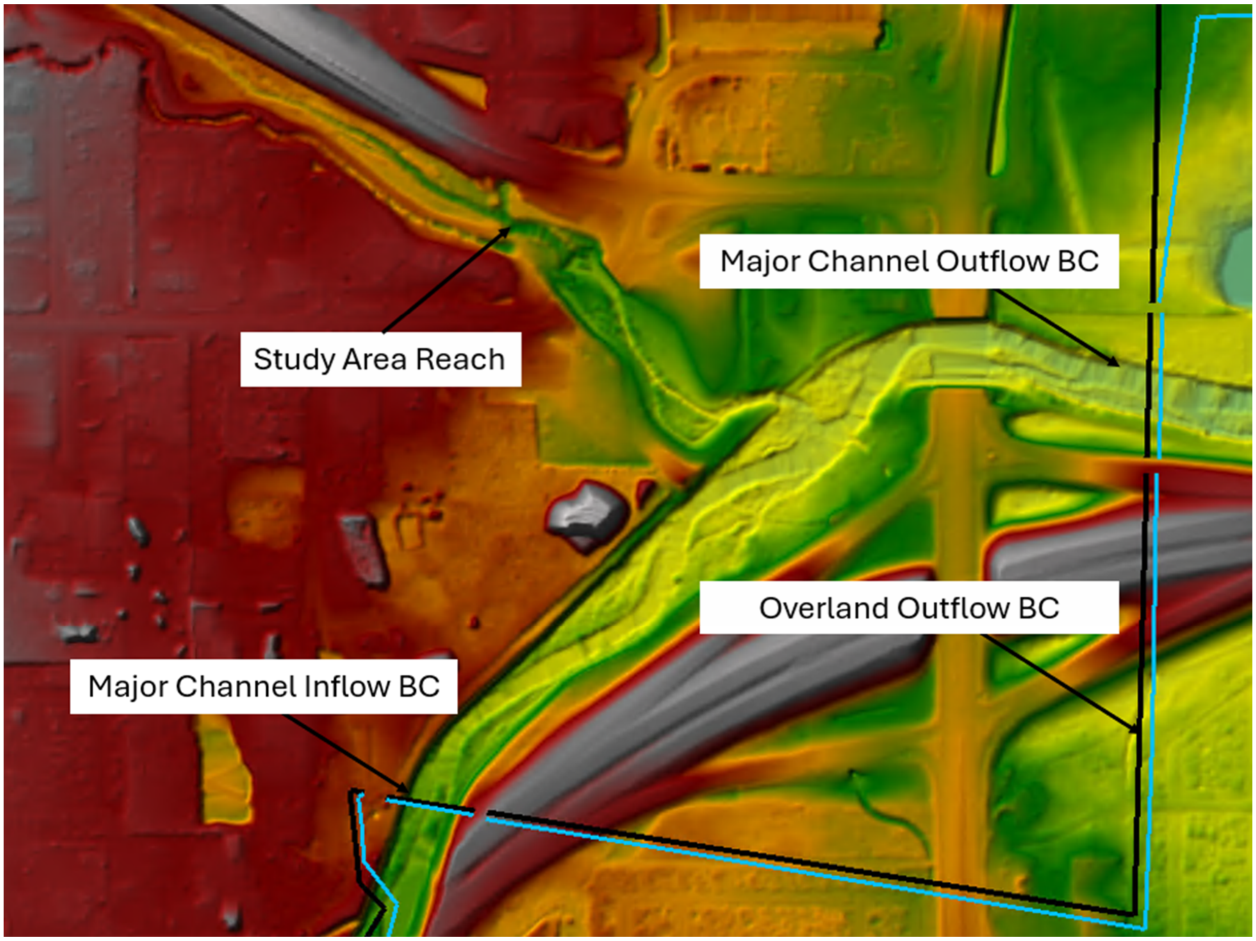


Figure 18. Example of Downstream Boundary Conditions at Major Channel Confluence

7 Geometry Construction

Mesh construction defines the computational mesh on which 2D hydraulic calculations are performed. This chapter covers nominal mesh sizing, channel and floodplain refinements, breakline enforcement, building alignment, and unrefinement strategies. A standardized approach to mesh construction is important for arriving at adequate output resolutions throughout the District in a manner that balances accuracy, stability, and run time.

7.1 Nominal Mesh

For urban areas it is recommended to use a 50-foot nominal mesh with 100-foot unrefinement regions to reduce total cell count in undeveloped regions or open space sections. In larger areas of lower development, a nominal mesh of 100 to 200-feet is recommended as it captures adequate accuracy and allows for lower run times. Based on HEC-RAS user guidance, cell sizes should avoid transitioning by more than 50% of cell area over a given region. It is recommended to add perimeter spacing or transition area refinement regions if necessary to improve calculations. Further reduction of cell sizes can be implemented along controlling features (channels, high ground, structures) to refine the hydraulics; however, this is not necessarily recommended as a blanket approach, but rather it should be based on accuracy and model stability.

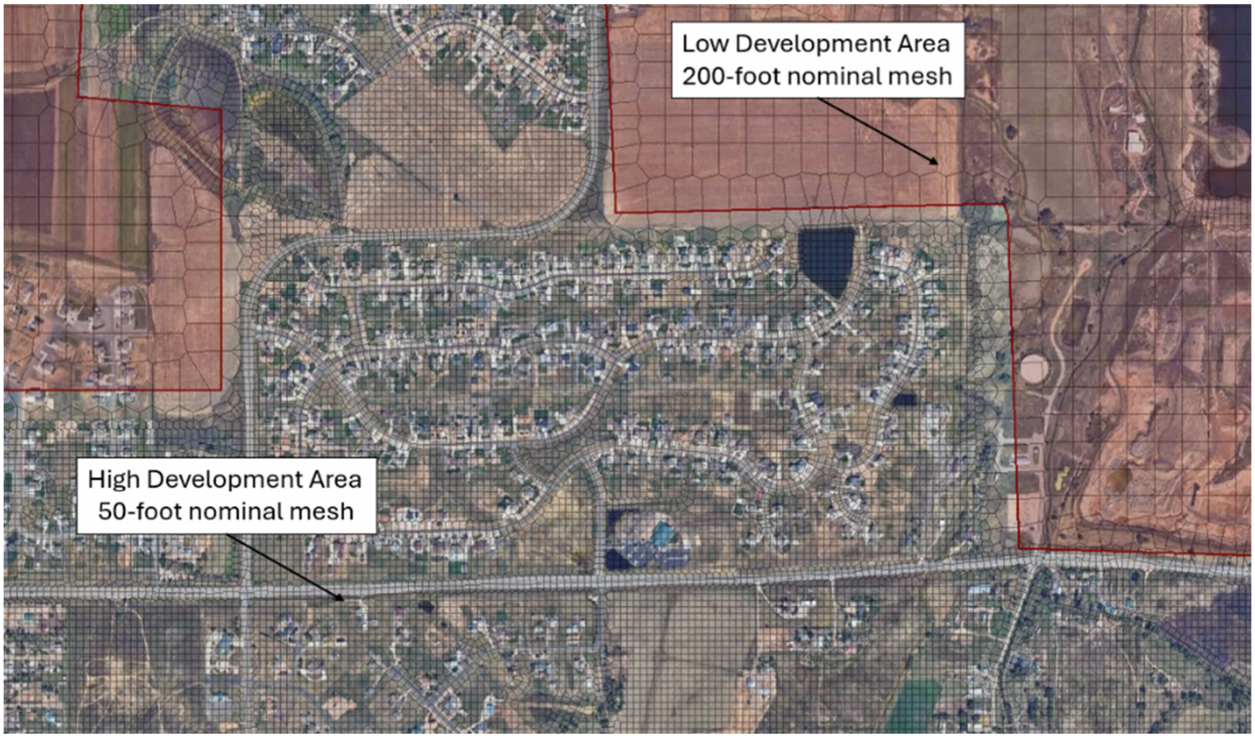


Figure 19. Nominal Mesh in High Development and Low Development Areas

7.2 Channel Refinements

Channel refinement regions should follow the thalweg of the channel and cover the area of interest in its entirety. Channel refinement regions should be enforced at a cell size equivalent to the channel width to align cell faces with the edge of channel banks with a minimum cell size of 25-feet. Channel banks, or edge of refinement regions should fall along the edges of the channel where terrain indicates a shift from channelized flow to overland flow. Figure 20 and Figure 21 illustrate refinement region edges along channel banks. Often channel refinement regions can be delineated by buffering a stream centerline by the average width of the channel. Where channel widths vary, refinement regions will need to be split, and the buffer distance and cell size should be adjusted accordingly. Refinement regions can be merged where there is overlap in regions and channel widths are the same. Channel refinement regions cannot be merged if two adjacent channels differ in width, as only one cell size can be set per refinement region.

In general, channel refinement cell sizes should be set so that one single cell is placed in the channel. However, in larger channels (>100 feet wide), multiple cells inside the channel should be used to avoid losing detail within the channel area. For example, for a 200-foot channel it is recommended to have 25 to 50-foot cells with smaller cell sizes preferred in areas with complex bathymetry, extreme slopes, and lower velocities. Figure 20 demonstrates an example of a 25-foot channel mesh refinement along Goldsmith Gulch and structures within a 50-foot nominal mesh. In this example the channel bottom was approximately 25 feet and cell sizes were set to 25 feet with 1 near repeat on either side. This was done to align more cell faces in the direction of flow and account for channel flow at full capacity.

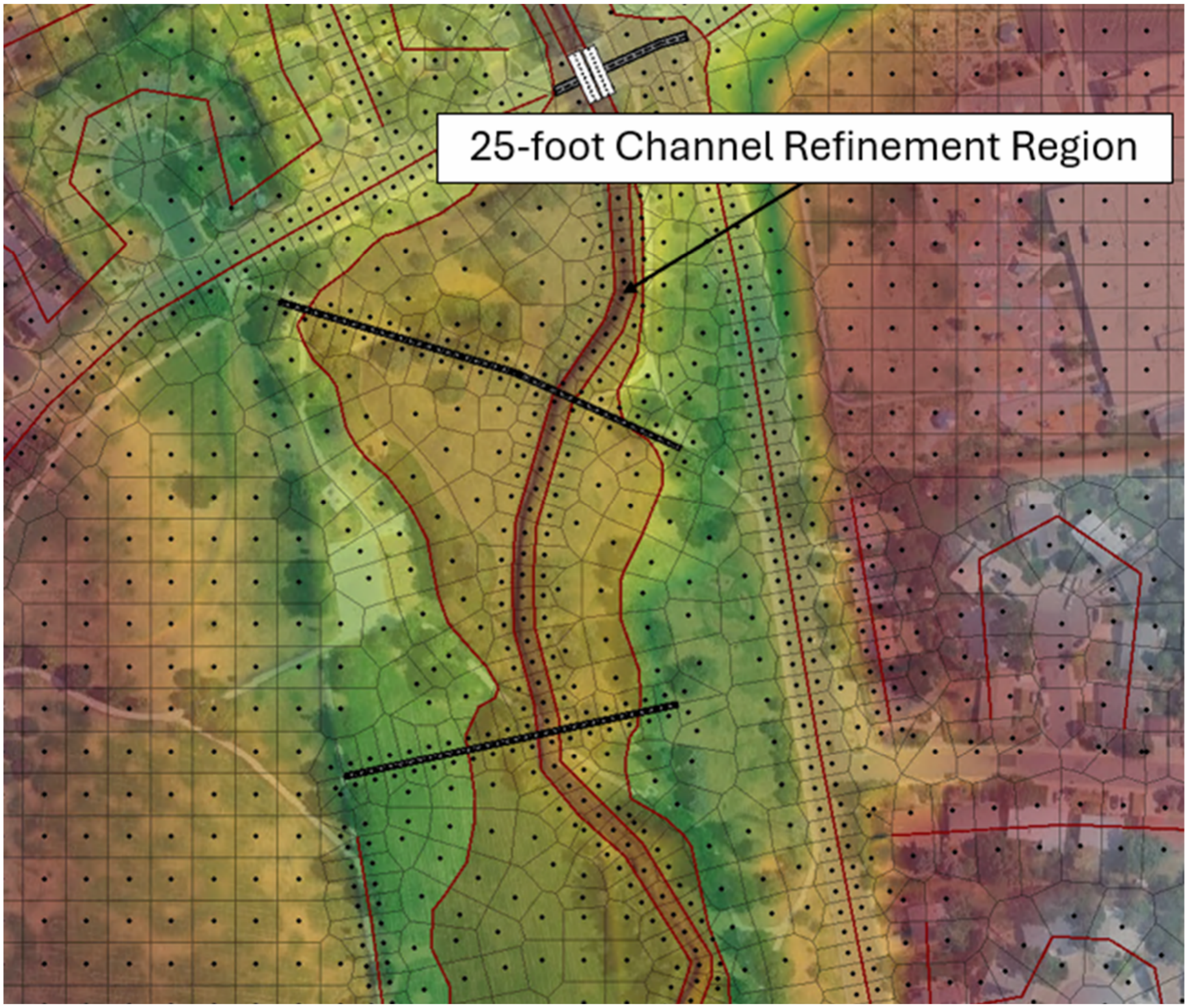


Figure 20. Channel Refinement Region Mesh Example

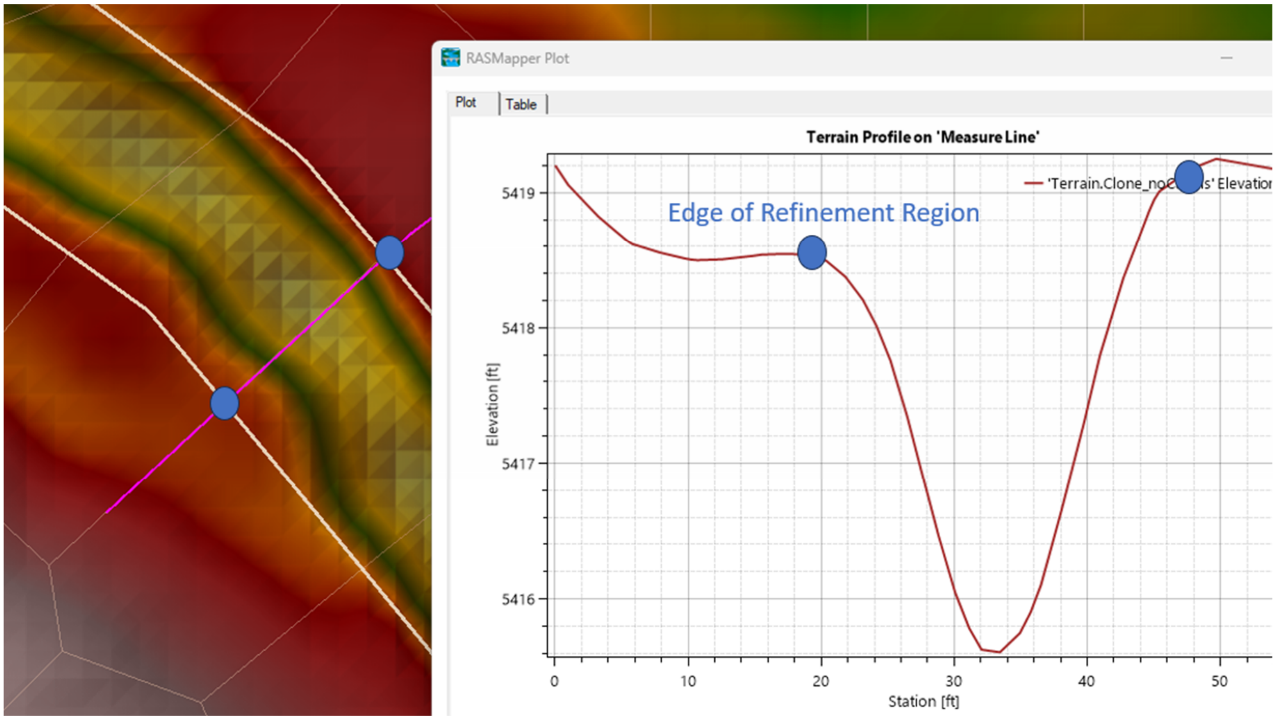


Figure 21. Channel Refinement Region Channel Banks Example

7.3 Floodplain Refinements

Floodplains must be enforced to a cell size of 25-feet or smaller for all models constructed within the MHFD urban areas. Smaller cell sizes are appropriate for narrow floodplains or high population centers. Floodplains should extend to the full width of the 1000-year fluvial floodplain up to the 130-acre drainage area threshold. The 1000-year fluvial floodplain may not be smooth in shape and therefore approximations and simplification of the floodplain refinement regions are encouraged. This process does not require iterative analysis. Once the base model is established, preliminary 1000-year results are acceptable for development of the floodplain region. It is best to buffer and simplify the region to reduce rendering times in RAS Mapper and to account for minor changes in the floodplain with additional refinement and structures.

Mesh sizes of 50 to100 feet have been deemed sufficient for open flat areas throughout the District that do not include areas of high-density development or are specifically areas of concern. This provides more accurate results by improving conveyance and detail in the calculations throughout the flooded area. Figure 22 demonstrates an example of a floodplain refinement region.

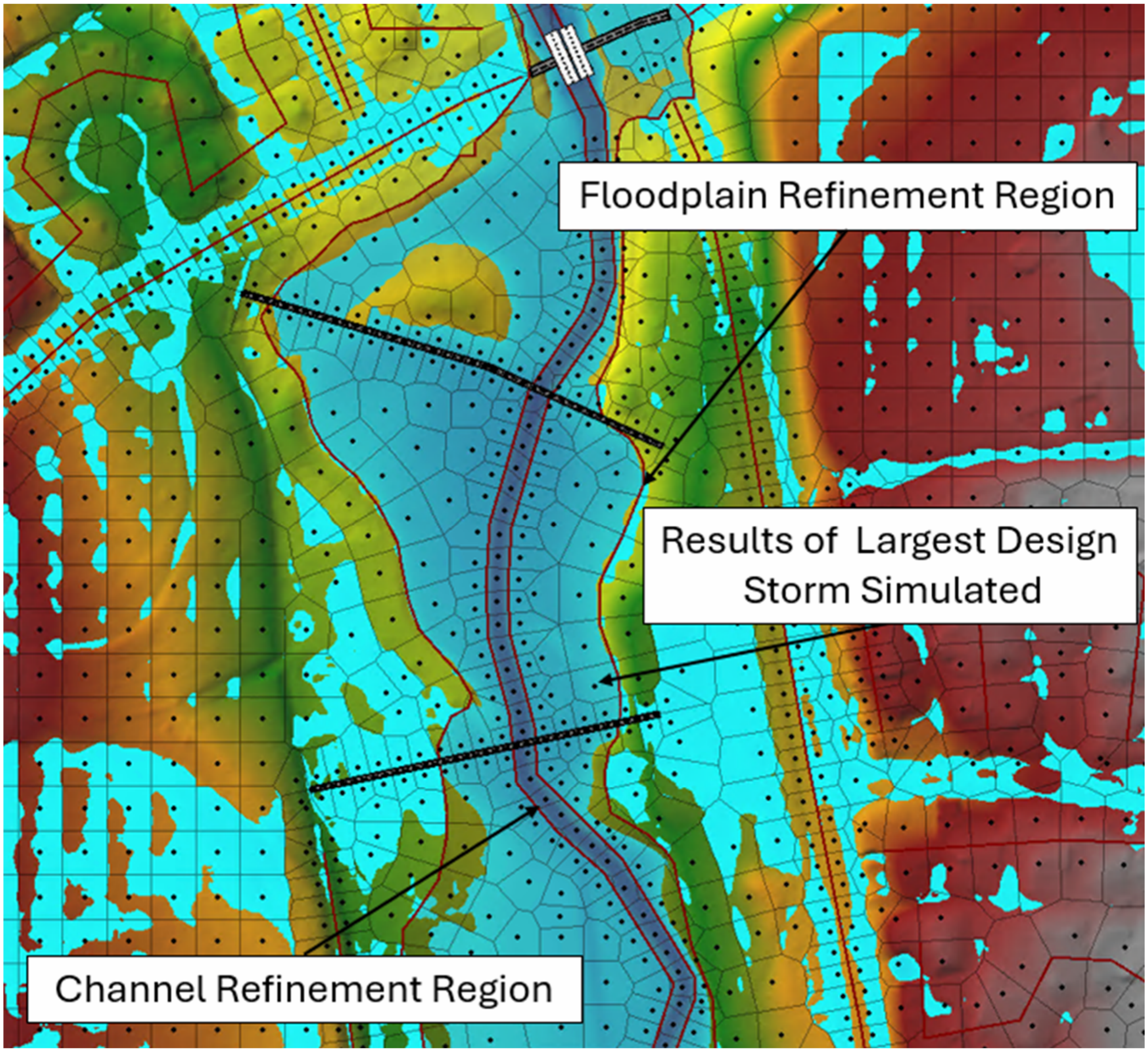


Figure 22. Floodplain Refinement Regions in an Urban Watershed

7.4 Breaklines

Breaklines should be enforced throughout the study area at a cell size equivalent to the floodplain refinement region or channel refinement region cell size depending on breakline location. Breaklines should ideally be enforced at a cell size of 25 to 50 feet in urban areas and 50 to 100 feet in undeveloped locations within the District. It is essential to be mindful of enforcement operation order for breaklines and refinement regions. It is recommended to first enforce refinement regions followed by breaklines and to avoid breaklines crossing refinement region boundaries where possible. Enforcement in the incorrect order may cause cell faces to not follow breaklines properly and may require manual intervention.

The following key topographic features warrant the use of properly enforced breaklines following the crown of the feature:

- *Roads/Railroads*: Compile national, state, and local transportation datasets to capture hydraulically significant roads and railroads. Many datasets may not align with the actual road ridges based on LiDAR, so review and edit breaklines to ensure major embankments and all stream crossings are accurately represented throughout the watershed. Many roadways may not actually be raised relative to the surrounding terrain. These roadways do not need to be represented with a breakline as there will be no measurable impact on simulation results.

- *Dam Embankments*: Add breaklines to all dam and pond embankments with drainage areas of at least 40 acres in urban areas and 160 acres in rural areas. For each dam embankment with a breakline, include a hydroconnector as a terrain modification unless it is a large flood-control dam with outlet or operational data available. In that case it would be represented with a SA/2D connection and rating curve based on as-built data or operational information.
- *Levees:* Breaklines should be added to all levees, whether accredited or non-accredited, using data from the National Levee Database (https://levees.sec.usace.army.mil/), which shows six levees within the District. Where embankment elevation data are available, apply a high ground terrain modification to reflect actual elevations, especially for floodwalls that are not visible in the terrain surface.
- *Berms and Non-Levee Embankments*: All berms and non-levee embankments that provide specific and meaningful hydraulic impacts should be represented with a breakline following the same protocol as roadways and railroads. These are assumed to remain intact during smaller flood events, but breach conditions should be evaluated for larger events using breach parameters via a simplified physical model inside of an SA/2D connection. Berms and non-levee embankments that fall outside of the significant areas of interest may be left without further refinement.
- *Jersey Barriers and Sound Walls:* Where jersey barriers or sound walls are present in a given watershed the local governments or MHFD should be consulted as to the proper procedure for hydraulic representation. It is not acceptable to assume complete hydraulic blockage from jersey barriers or sound walls without consultation.
- *Buildings:* Buildings must be represented with breaklines as further outlined in Section 7.5.
- *Lakes and Reservoirs*: Place breaklines along large lake and reservoir perimeters to properly account for hydraulics and storage along the boundary.
- *Natural High Ground*: Include breaklines for significant natural high points such as ridges between floodplain areas. These features help maintain accurate terrain modeling and flow paths.

7.5 Buildings

Cell faces must be aligned with buildings in the land cover layer for their impacts to be properly represented in simulation results. This is done by enforcing breaklines along buildings at a cell size equal to the surrounding nominal, urban, or floodplain mesh size, as illustrated in Figure 23.

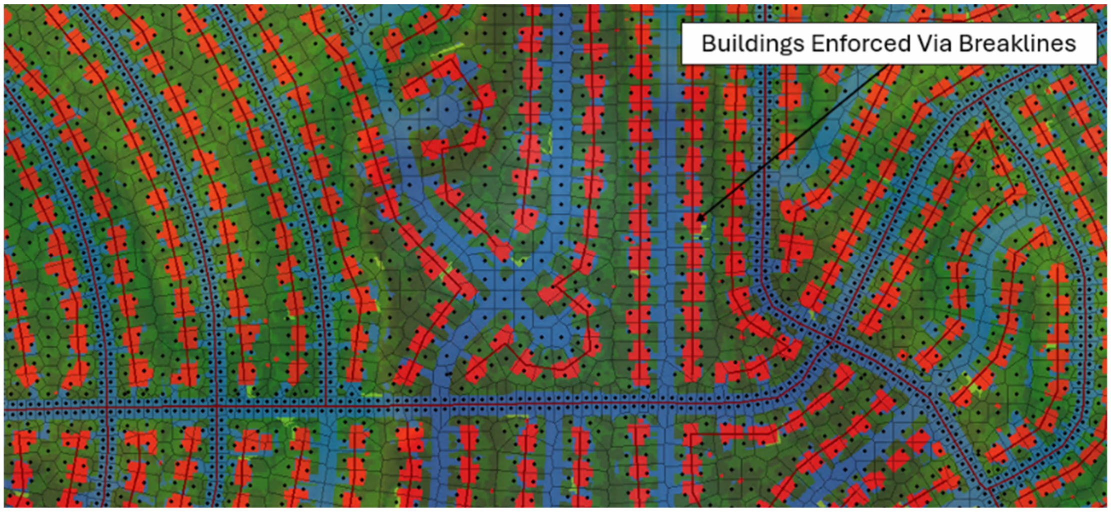


Figure 23. Example of Buildings Enforced with Breaklines in an Urban Area

7.6 Unrefinement

Refinement regions also allow for the user to apply larger cells than the surrounding mesh, which in this document is referred to as “unrefinement”. Unrefinement regions should be used for large undeveloped areas without much terrain variability, such as reservoirs, golf courses, and some open-space or greenways. This allows for increased cell size to reduce the total cell count for improved computation time. Mesh sizes for these undeveloped areas can range from 100 to 200 feet within the District and 200 to 400 feet for cells outside of the District that are only utilized for hydrologic routing purposes.

For larger hydro-flattened lakes or reservoirs, unrefinement region should align with the banks of the reservoir along the hydro-flattened surface (or the depth-modified surface if altered to match normal pool elevation) to not over reduce accuracy in calculations beyond the hydro-flattened area. Typically, these cells within reservoirs will be 200 to 400 feet. Figure 24 demonstrates an example of a lake captured with a unrefinement region.

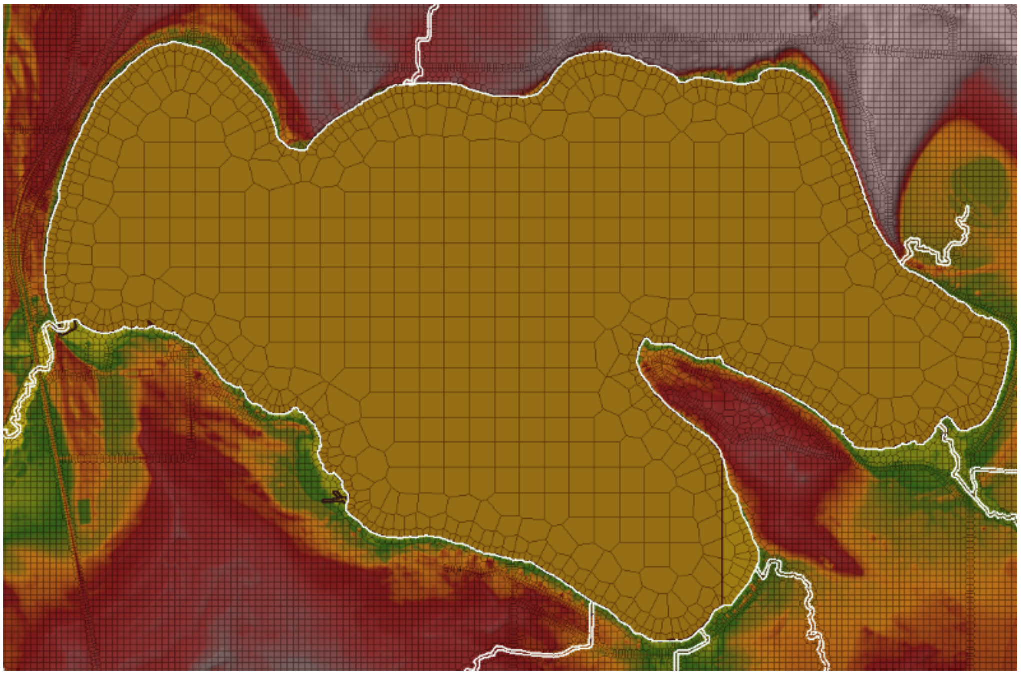


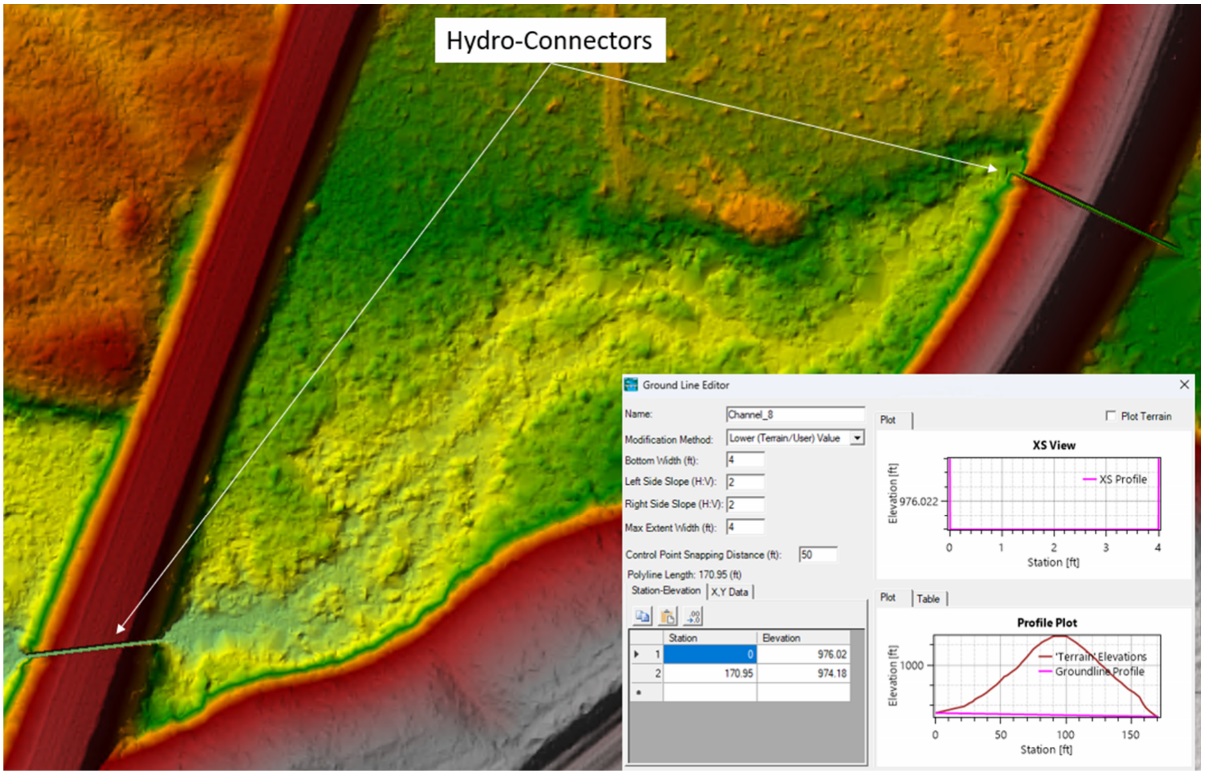
Figure 24. Hydro-Flattened Lake with Unrefinement Region to Reduce Cell Count

8 Structures and Stormwater Infrastructure

Structures and stormwater infrastructure play a pivotal role in urban flood modeling, affecting conveyance, ponding, and drainage. This chapter introduces methods for representing hydro-connectors, bridges, culverts, reservoir outlets, and stormwater pipes within the model. Standardized pre-processing and attribute assignments assist in the accurate integration of controlling features which supports consistent hydraulic performance across projects.

8.1 Hydro-Connectors

All hydraulic structures within the 130-acre drainage areas with survey-based information or field measurements must be represented with SA/2D connections. However, there are frequent instances where smaller structures along minor drainages are not surveyed or captured in prior models. In such cases, hydro-connectors via terrain modifications may be used to approximate structures for conveyance if detailed structure data are not available. Hydro-connectors can be frequently applied to smaller structures and pipes in the overbanks and throughout developed portions of the watershed outside of the channel. Hydro-connectors are applied as terrain modifications that approximate structures for conveyance and should be applied with a reasonable approximation of width based on the bridge or culvert opening and its endpoint elevations matching the channel upstream and downstream of the structure.



Name: Channel_8
Modification Method: Lower (Terrain/User) Value
Bottom Width (ft): 4
Left Side Slope (H:V): 2
Right Side Slope (H:V): 2
Max Extent Width (ft): 4
Control Point Snapping Distance (ft): 50
Polyline Length: 170.95 (ft)
Station-Elevation | X,Y Data

	Station	Elevation
1	0	976.02
2	170.95	974.18

Figure 25. Terrain Modifications Used to Approximate Structures for Lower Levels of Detail

8.2 Bridges and Culverts

SA/2D connections are to be used to represent structures in study areas. Structure geometry must be sourced from accurate and relevant reference materials. Often this refers to effective Zone AE or FHAD modeling, as-built information, local government databases, or surveyed information. It's recommended to reach out to local governments or the District, where appropriate, to ensure the highest quality and time-relevant reference material are sourced. Where updated terrain is available compared to the effective models, stationing can be adjusted slightly to structure openings in the 1D to 2D conversion, but shifts should be kept to a minimum if possible. All structure sources must be added to the structure geometry description window to clearly identify the reference material used for structure development (e.g., effective 1D model, spatial survey data, as-builts, field measurements, or desktop approximation). Explicitly list the names and dates of reference material for future model use clarity. Please refer to the "Hydraulic Structures Inside of 2D Flow Areas" section of the HEC-RAS User's Manual for bridge parameter and cell alignment guidance for bridge structures.

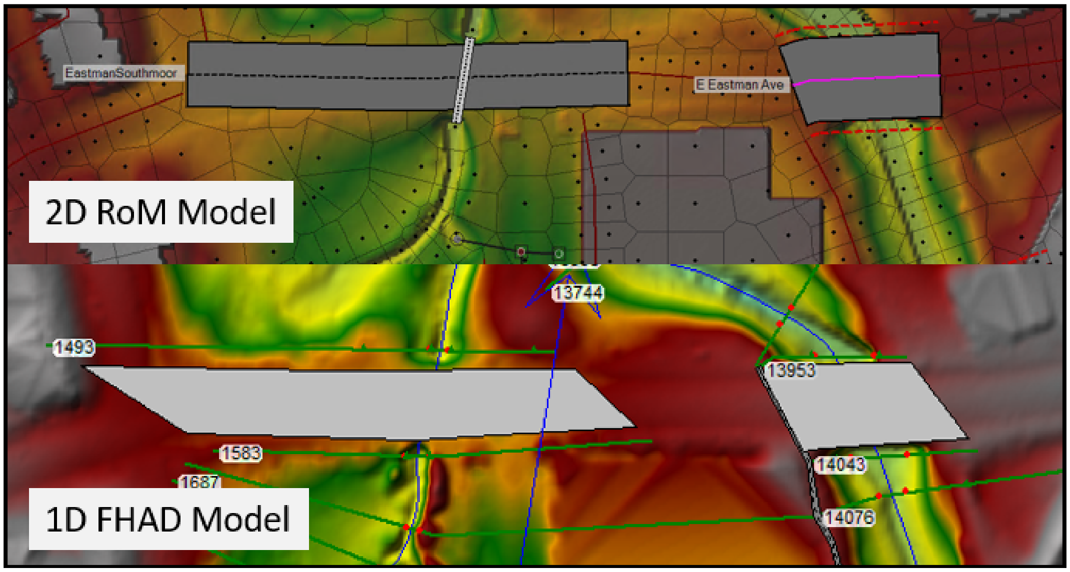


Figure 26. Structure Geometry Comparison Between 2D RoM and FHAD at Eastman Ave

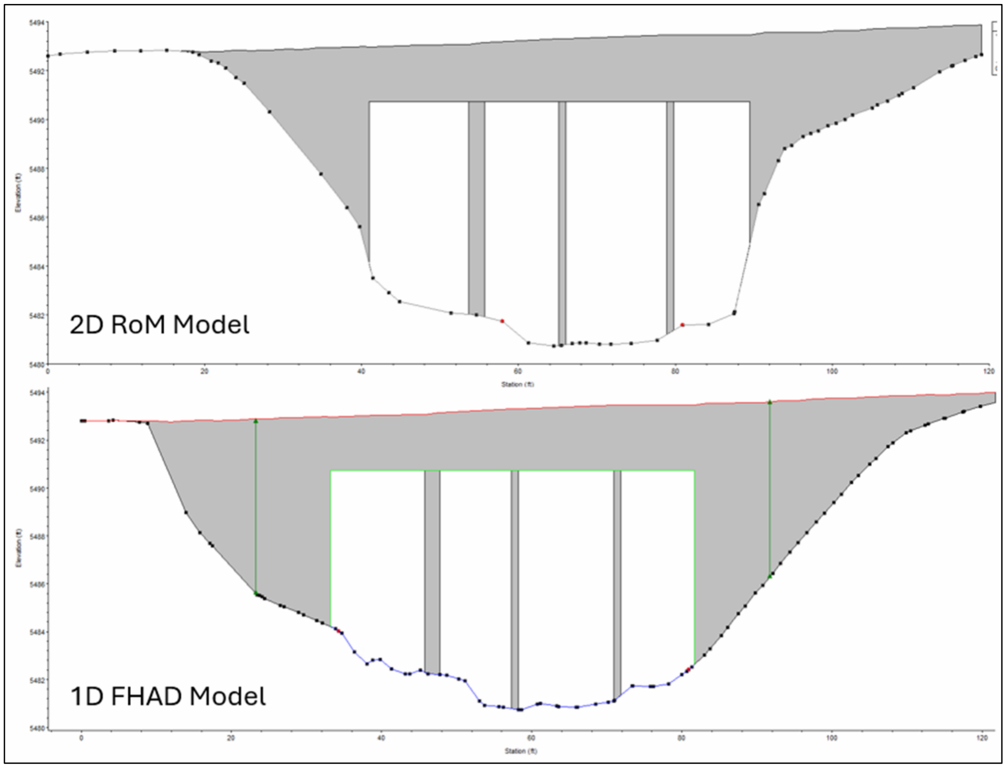


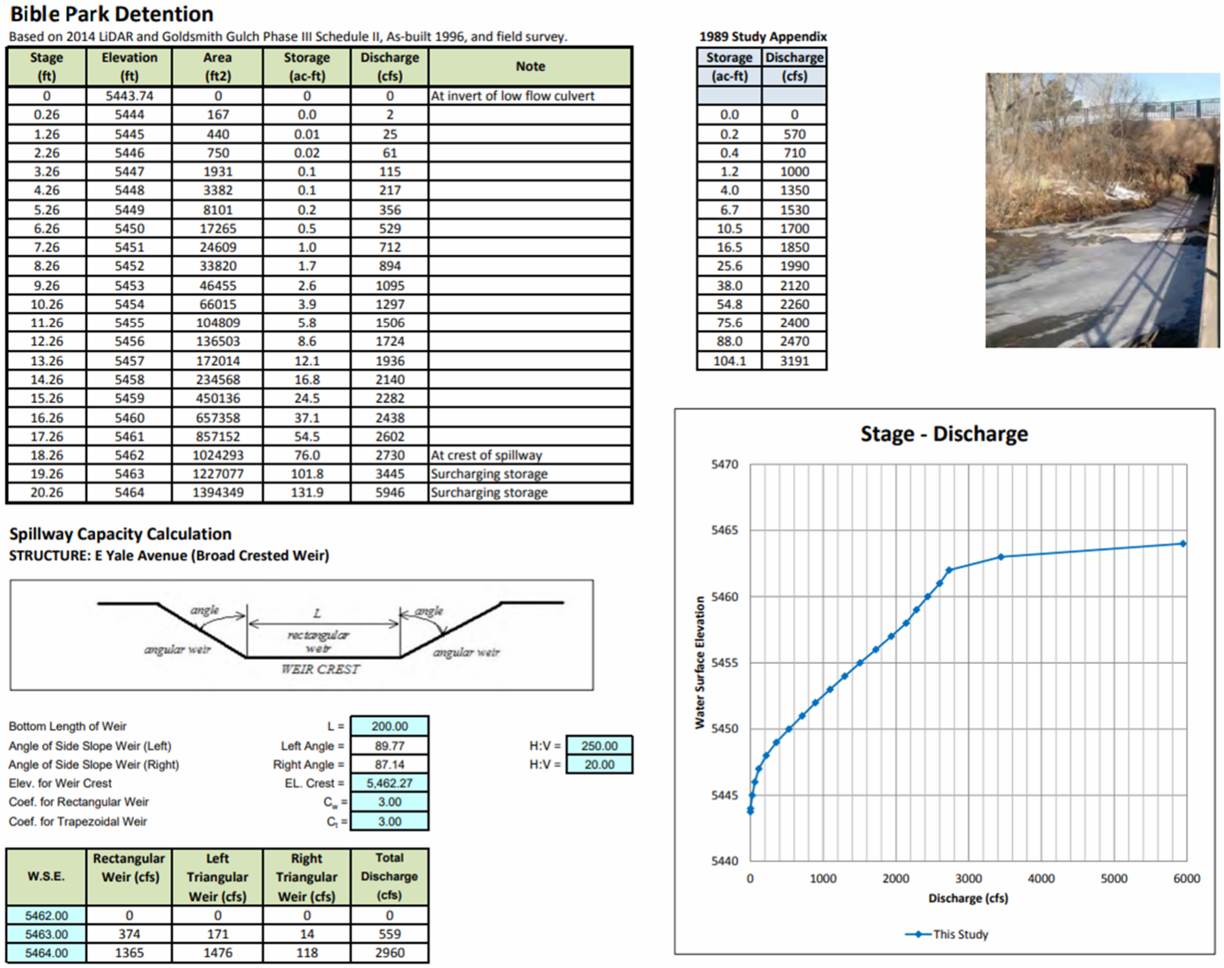
Figure 27. Example of Shifted Structure Stationing from 1D FHAD to 2D Model

Entrance and loss coefficients selected for culvert geometries should be obtained referencing table 11-3 of the MHFD USDCM volume 2. For bridges a standard weir coefficient of 2.6 is recommended by HEC-RAS for flow over roadways and railings. A coefficient of 3.0 is recommended for flow over elevated roadway approach embankments where applicable.

8.3 Reservoir and Detention Pond Outlets as Rating Curves

Where rating curve information is available for reservoirs or detention facilities, it is the preferred method to represent outlet structures to maintain consistency between District products. Outlets should be represented by SA/2D connections with manual rating curves programmed into the HEC-RAS geometry using District or community outlet and datum information. Figure 28 and Figure 29 demonstrate an example where the Bible Park Detention rating curve was translated from the Goldsmith Gulch FHAD report into the 2D RoM model for risk assessment.

It is important to maintain consistency and translate datums between the FHAD or as-built rating curves and the 2D RoM rating curve, where necessary, as calibration can be heavily influenced by rating curve parameters. The drawing and reports should also be consulted to determine inlet and outlet locations as they may not always be obvious from satellite or terrain. All structure and rating curve sources must be added to the structure geometry description window to clearly identify the reference material used for structure development.



Bible Park Detention

Based on 2014 LiDAR and Goldsmith Gulch Phase III Schedule II, As-built 1996, and field survey.

Stage (ft)	Elevation (ft)	Area (ft2)	Storage (ac-ft)	Discharge (cfs)	Note
0	5443.74	0	0	0	At invert of low flow culvert
0.26	5444	167	0.0	2	
1.26	5445	440	0.01	25	
2.26	5446	750	0.02	61	
3.26	5447	1931	0.1	115	
4.26	5448	3382	0.1	217	
5.26	5449	8101	0.2	356	
6.26	5450	17265	0.5	529	
7.26	5451	24609	1.0	712	
8.26	5452	33820	1.7	894	
9.26	5453	46455	2.6	1095	
10.26	5454	66015	3.9	1297	
11.26	5455	104809	5.8	1506	
12.26	5456	136503	8.6	1724	
13.26	5457	172014	12.1	1936	
14.26	5458	234568	16.8	2140	
15.26	5459	450136	24.5	2282	
16.26	5460	657358	37.1	2438	
17.26	5461	857152	54.5	2602	
18.26	5462	1024293	76.0	2730	At crest of spillway
19.26	5463	1227077	101.8	3445	Surcharging storage
20.26	5464	1394349	131.9	5946	Surcharging storage

1989 Study Appendix

Storage (ac-ft)	Discharge (cfs)
0.0	0
0.2	570
0.4	710
1.2	1000
4.0	1350
6.7	1530
10.5	1700
16.5	1850
25.6	1990
38.0	2120
54.8	2260
75.6	2400
88.0	2470
104.1	3191

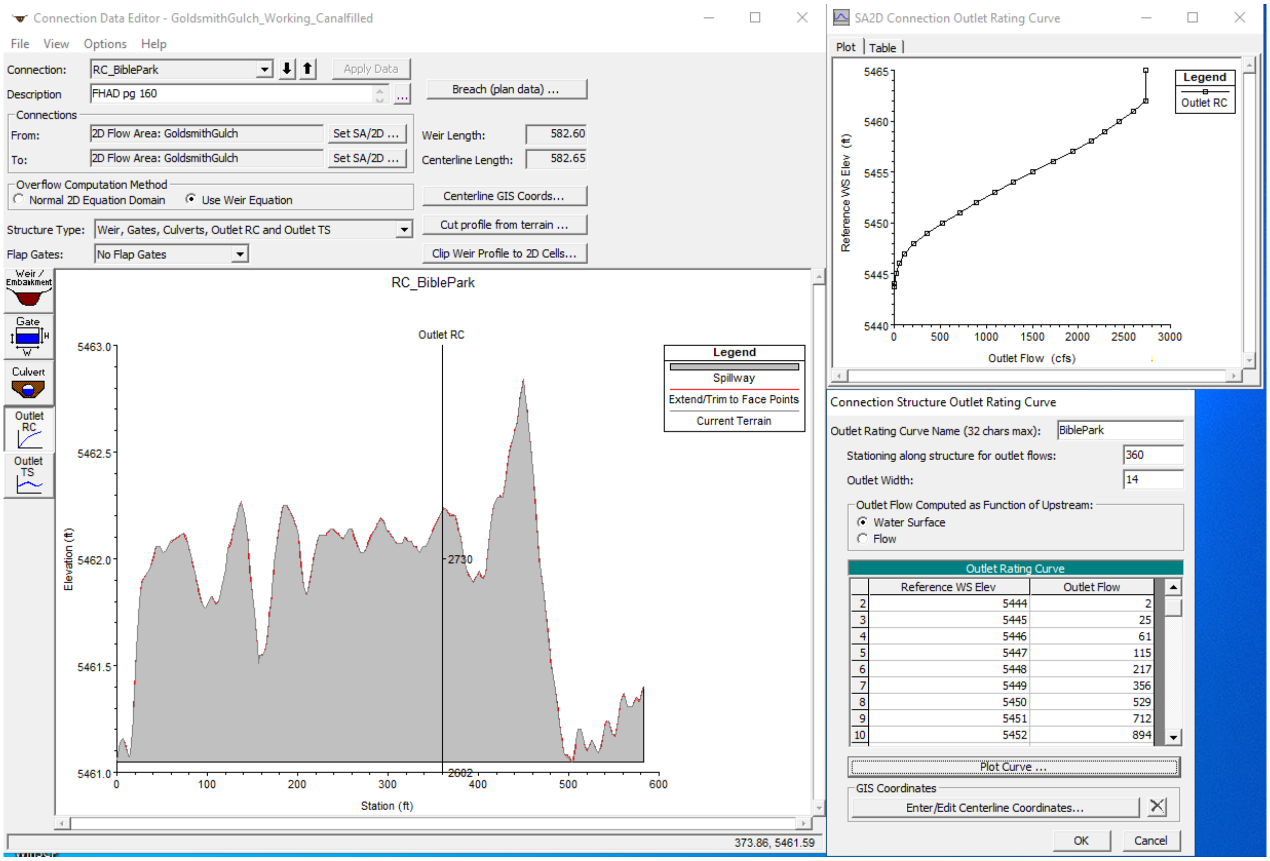
Spillway Capacity Calculation

STRUCTURE: E Yale Avenue (Broad Crested Weir)

Bottom Length of Weir	L =	200.00		
Angle of Side Slope Weir (Left)	Left Angle =	89.77	H:V =	250.00
Angle of Side Slope Weir (Right)	Right Angle =	87.14	H:V =	20.00
Elev. for Weir Crest	EL. Crest =	5,462.27		
Coef. for Rectangular Weir	C_w =	3.00		
Coef. for Trapezoidal Weir	C_t =	3.00		

W.S.E.	Rectangular Weir (cfs)	Left Triangular Weir (cfs)	Right Triangular Weir (cfs)	Total Discharge (cfs)
5462.00	0	0	0	0
5463.00	374	171	14	559
5464.00	1365	1476	118	2960

Figure 28. Example Rating Curve Information Typically Presented in a FHAD Report



Connection: RC_BiblePark

Description: FHAD pg 160

Connections

From: 2D Flow Area: GoldsmithGulch

To: 2D Flow Area: GoldsmithGulch

Weir Length: 582.60

Centerline Length: 582.65

Overflow Computation Method: ○ Normal 2D Equation Domain ● Use Weir Equation

Structure Type: Weir, Gates, Culverts, Outlet RC and Outlet TS

Flap Gates: No Flap Gates

Connection Structure Outlet Rating Curve

Outlet Rating Curve Name (32 chars max): BiblePark

Stationing along structure for outlet flows: 360

Outlet Width: 14

Outlet Flow Computed as Function of Upstream:
● Water Surface
○ Flow

Outlet Rating Curve

	Reference WS Elev	Outlet Flow
2	5444	2
3	5445	25
4	5446	61
5	5447	115
6	5448	217
7	5449	356
8	5450	529
9	5451	712
10	5452	894

Figure 29. Example Rating Curve Translated into HEC-RAS Geometry

Note that in select instances where a detention pond was recently installed and upstream development has not yet meaningfully taken place, a large hydroconnector should be incorporated through the detention pond embankment to prevent storage or flow attenuation. The reason is that in near future, upstream development will increase runoff that will be attenuated by the pond, but in the interim, it would underestimate downstream flooding to attenuate the lower flows from upstream runoff prior to development.

8.4 Canals

Canals within the District should be modeled as not providing conveyance or storage during design events. This can be achieved through intentionally leaky cells allowing water to pass over berms on either side of the canal system, through terrain processing to consume storage, or through terrain modifications used to fill canals up to the berm elevation. For calibration events, canal storage may be represented more realistically if supported by historical information describing canal conditions, or if excluding storage adversely affects calibration performance. However, because the initial water level in canals is often uncertain, any departure from the no-storage assumption should be discussed with the District PM and clearly documented.

8.5 Stormwater Infrastructure

Stormwater pipes can be modeled using the standard built-in pipe features available in HEC-RAS Version 7.0 and subsequent releases. The latest dataset for storm sewer pipes and nodes (inlets, manholes and outlets) should be acquired for the study area in coordination with the District and local governments (Figure 31). Before integrating storm sewer pipes and nodes (inlets and outlets) into a model, the data need to be pre-processed to meet storm sewer modeling standards. The following checks are recommended for consistency, hydraulic accuracy, and compatibility with modeling workflows.

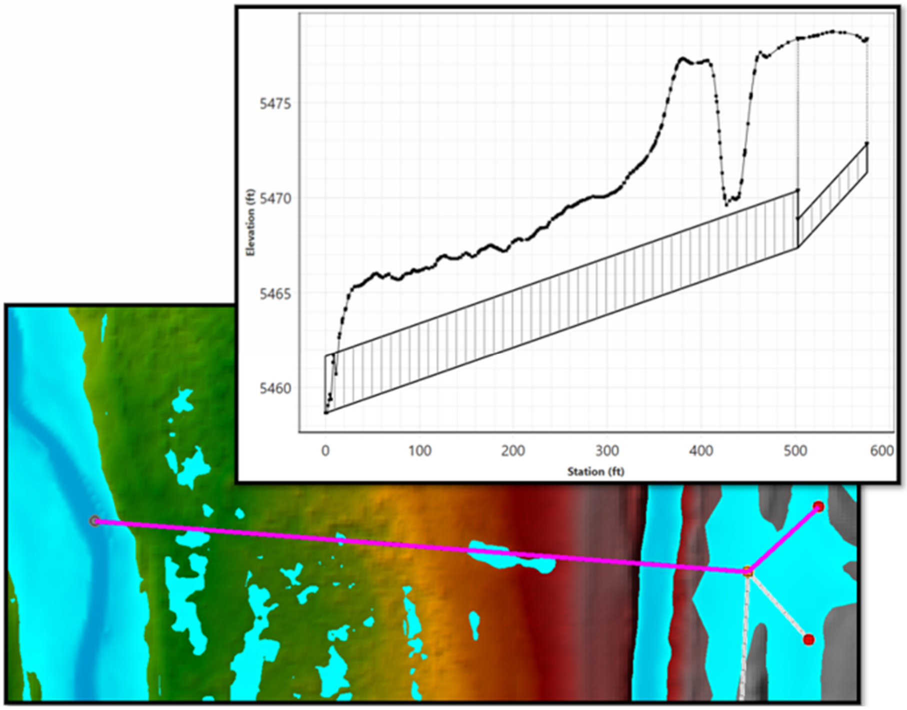


Figure 30. Profile Plot of Conduits Selected in Pink with Two Drop Inlets and an Outlet

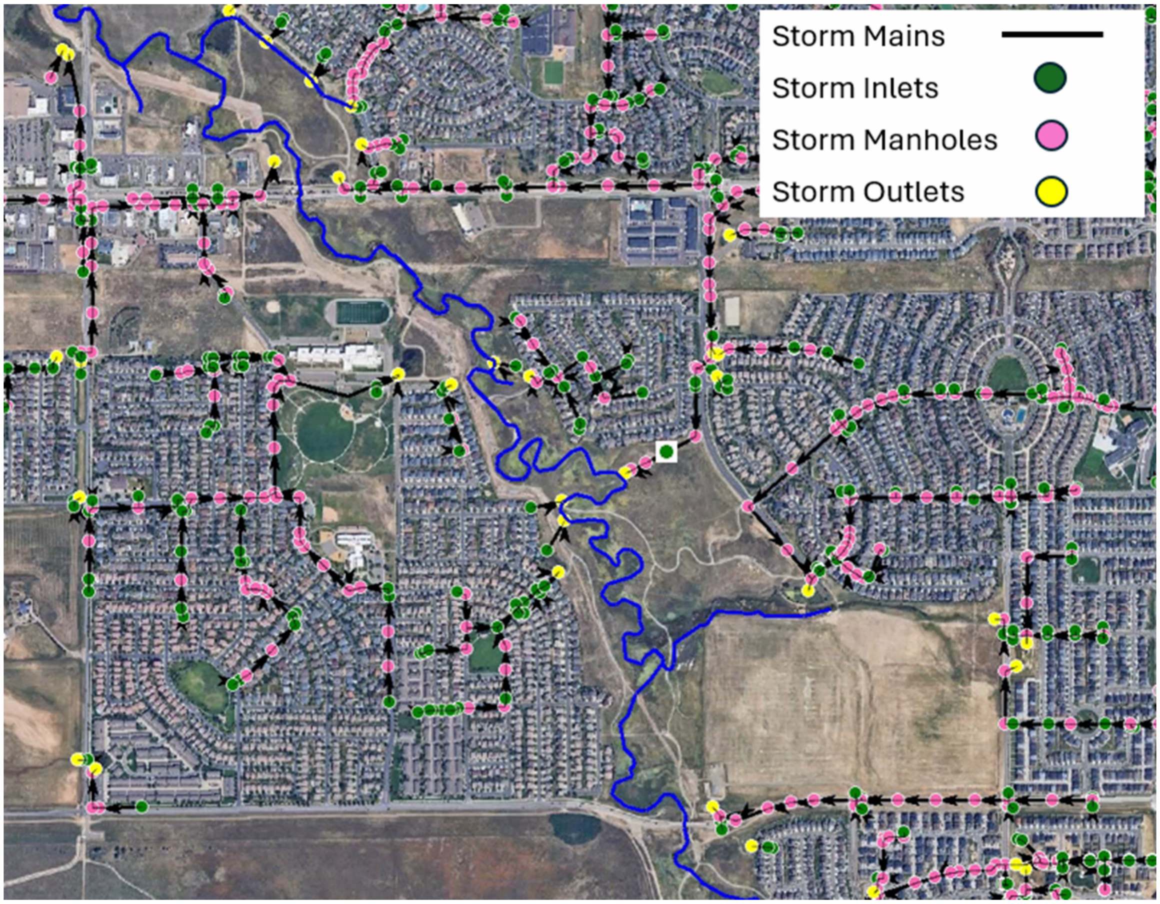


Figure 31. Example Stormwater System GIS Data

8.5.1 Pre-Processing Checks for Inlets, Outlets, and Pipes

Often the geospatial data received on pipe systems are poorly attributed and contain network errors concerning alignments, snapping, directions, and outlets. The following steps should be taken to pre-process the pipes before incorporating them into the hydraulic model.

1. Verify Vertical Datum Consistency

Invert elevations from the as-received pipe and node dataset should be verified to ensure consistency with the model vertical datum. Legacy datasets may use NGVD29 and should be converted to NAVD88 where needed to align with terrain elevations.

2. Apply Diameter Thresholds Based on Analysis Type

All pipes that are geospatially represented (digitized in a useable schema) will be incorporated into HEC-RAS as pipes. Storm sewer information that is not readily ingestible, such as in as-builts, PDFs, or imagery, will not be incorporated without specific instruction from MHFD due to the level of effort for capture.

While prior versions of HEC-RAS (e.g., 6.7) were restricted to singular pipe representations, HEC-RAS 7 does allow for multiple parallel pipes, which is the preferred default approach. Should there be a location-specific reason for consolidating multiple parallel pipes into a simplified single pipe, the Manning’s equation can be used to match total conveyance capacity, meaning that the flow is preserved under the same head or energy gradient. The combined conveyance of the parallel pipes must nearly equal the conveyance of the replacement pipe. If slope and roughness are consistent, the equivalent diameter is notably smaller than the sum of the individual diameters (e.g., three 18-inch pipes have approximately the same capacity as a hypothetical single 27-inch pipe).

The simplified equation to calculate an equivalent bigger pipe from multiple parallel pipes is given as $D_{eq} = D \times N^{3/8}$, where N is the number of pipes being aggregated, D is the diameter of each pipe in feet and D_{eq} is the equivalent diameter with similar conveyance capacity. More details on derivation of this simplified equation can be found from this American Concrete Pipe Association Publication. Where possible, multiple parallel pipes should be modeled as such to more realistically reflect the actual configuration.

3. Invert and Manhole Elevations Should be Below Ground

For all nodes except outlets, the invert elevation should be set no higher than the ground surface elevation minus the sum of the pipe diameter, pipe wall thickness (default of 0.25 foot), and a free cover buffer (default of 0.25 foot unless specified for the study area). This ensures pipes are realistically placed underground and not exposed above the surface.

4. Upstream Elevation Should be Greater than Downstream

For each pipe segment, the invert elevation at the upstream node must be greater than at the downstream node. If this condition is not met in the source data, the downstream invert must be adjusted to be lower than, or at minimum equal to, the upstream invert. Any configuration that maintains a downstream slope is acceptable, provided it produces stable conduit hydrographs. These adjustments are applied to support model stability and standardization and do not imply that the original survey or source inverts are necessarily incorrect. When multiple pipes connect at the same junction, invert elevations are not required to be identical. HEC-RAS allows pipes to enter and leave a node at different elevations using upstream and downstream offsets, which define the conduit invert relative to the node invert.

5. Pipe Profile Must Stay Below Surface at all Points

Intermediate nodes along pipe segments are checked to verify that the pipe profile remains below the ground surface at all points and is consistent with the surface elevation rule. Conduits that do not meet this condition are flagged for review, primarily to identify potential anomalies in the terrain data or pipe GIS dataset that may be causing unrealistic elevations. There is no expectation that all flagged cases must be corrected or forced to meet this condition. *HEC-RAS can still run with these conditions, while InfoWorks ICM will generate warnings*.

6. Check ponded urban areas for missing storm sewer features

After initial simulations, additional inlets may be added only where clearly identifiable in aerial imagery and where the storm sewer GIS dataset provided by the community or District seems to be incomplete. Additions should be limited to areas with heavily impacted receptors such as roadways or buildings experiencing substantial or prolonged ponding that disrupts access or function or increases damage potential.

Pipe sizing should remain conservative to avoid overestimating conveyance. An 18-inch pipe may be used when tying into an existing subsystem, while a 24-inch pipe may be used only if modeled as a separate subsystem. For highly critical areas, coordinate with the District and stakeholders before making changes to allow for field verification or use of additional data such as as-builts, site photos, or survey information.

7. Verify Outfall Segments for Realistic Drainage

Outfalls must maintain a positive slope (upstream > downstream to ensure proper flow, but should also be verified against aerial imagery when notably altered to confirm they haven't been lowered beyond realistic limits. At minimum, the pipe opening should remain above ground to allow free drainage.

Outfall locations may also require inspection against aerial imagery to validate these conditions. In some cases, manually shifting the location may be required to achieve a more realistic hydraulic discharge condition.

8. Verify Pipe Slope Reasonableness

SEMSWA Stormwater Manual Section 9.8.1 indicates that storm sewer systems should be designed to achieve velocities between about 4 and 18 fps, which are strongly influenced by pipe slope. To support these conditions, they recommend minimum slopes of roughly 0.5% for pipes 30 inches and larger, and a minimum slope of 1% for smaller pipes. For the MHFD Flood Risk Program, these slope ranges are used as practical reference targets rather than strict constraints. In many cases, invert elevations are forced to be interpolated, making it difficult to meet the recommended slopes and resulting velocity ranges while also satisfying other system constraints, for example preprocessing checks 3, 4, 5 and 7, at the same time.

8.5.2 Attribute Population in RAS Mapper

After pre-processing the nodes and conduits, the data must be imported into RAS Mapper, where corresponding attributes should be populated. The modeler may choose to populate these attributes prior to import using GIS or an automated workflow, or post-import directly within the attribute tables in RAS Mapper.

The following parameters should be set in the Conduit and Node attribute tables in accordance with the guidance below. Table 9 and Table 16 show the specific attribute tables to be edited for Conduits and Nodes, respectively. Definitions and explanations for each parameter are provided in the Pipe Networks chapter of HEC-RAS User's Manual, while detailed descriptions of how these parameters are used to compute pipe system hydraulics are documented in the Modeling Pipe Networks chapter of HEC-RAS Reference Manual.

8.5.2.1 Conduits

Rise and Span

The sizing method should follow a single, consistent decision flow so different reviewers processing the same data will reach identical final results. The goal is to preserve reliable source information, apply conservative assumptions only when needed, and produce outputs that are transparent and reproducible.

In this process, diameter is a single representative size value used for invert elevation and cover calculations. For circular pipes, diameter, width, and height are all equal. For non-circular sections (boxes, ellipses, etc.), diameter is set equal to the height dimension.

Prioritize Source Data

The first and strongest principle is to use valid source dimensions as the primary basis for final sizing. Source data carries field observation or design authority and should be kept intact whenever it is complete and plausible. This means if both width and height are present and greater than zero, those values are carried forward together. If a valid diameter is directly available from source data (for example, a recorded circular pipe diameter), that diameter remains the preferred value. Only when source data is incomplete or missing do assumptions come into play.

Missing One Dimension: Mirror to Maintain Consistency

When either width or height is present, but its counterpart is missing, the solution is to mirror the available dimension to ensure internal consistency. If width is available but height is missing, set height equal to width up to a maximum of 24 inches; if width exceeds 24 inches, cap the inferred height at 24 inches. Conversely, if height is present but width is missing, set width to the available height.

The reasoning for this approach is twofold. First, without explicit data for both dimensions, assuming a square or roughly circular section (1:1 aspect ratio) is more conservative and realistic than leaving dimensions mismatched. Second, the 24-inch cap on width-only records prevents them from driving implausibly large, inferred heights, which would inflate storage capacity and exaggerate invert and cover calculations downstream. Diameter is then set equal to the finalized height value.

No Usable Dimensions: Apply Network-Based Defaults

When both width and height are missing or invalid, pipe size cannot be assigned directly. Instead, apply a network-based fallback using information from the connected system. Use the following hierarchy:

- *No known sizes in the system*: Assign 18 inches to all pipes to provide a conservative size that is more likely to underestimate conveyance.
- *Some sizes exist in the system*: Assign the largest known upstream size to pipes with missing dimensions, where a direct upstream connection exists. This reflects typical gravity system behavior, where pipe sizes increase downstream as flow accumulates, and grounds the assignment in observed system conditions.
- *No upstream evidence for a specific pipe (but sizes exist elsewhere in the subsystem)*: Assign 18 inches. This leverages a conservative size while still keeping the subsystem in play.

Record Provenance: Enable Audit and Reproducibility

To make results reproducible and reviewable, every finalized diameter value carries a provenance tag in the zDIAMETER field indicating its origin. Use the following standardized tags:

- PRVD (Provided): The diameter value came directly from the source shapefile (PIPE_DIAMETER_FIELD) and was retained because it was non-zero and plausible.
- SYN (Synthesized): The diameter was derived from available width and/or height dimensions when the explicit diameter field was missing or zero
- ASM (Assumed): The diameter was assigned using standardized defaults when neither an explicit diameter nor usable width/height dimensions were available.

Table 7. Example Calculations for the Final Diameter Value used in Invert Calculations and Pipe Dimension Assignments

Case	Source Diameter	Source Width / Height	Upstream Sized Path	Final Diameter	Final Width / Height	Flag
Source diameter present	30"	—	n/a	30"	30 × 30	PRVD
No diameter, both dimensions present	—	30 W × 24 H	n/a	24"	30 × 24	SYN
No diameter, width only, width ≤ 24"	—	18 W	n/a	18"	18 × 18	SYN
No diameter, width only, width > 24"	—	36 W	n/a	24"	36 × 24	SYN

Case	Source Diameter	Source Width / Height	Upstream Sized Path	Final Diameter	Final Width / Height	Flag
No size in connected subsystem	—	—	No	18"	18 × 18	ASM
Some size exists, upstream path found	—	—	Yes, max upstream 42"	42"	42 × 42	ASM
Some size exists, no upstream path	—	—	No	18"	18 × 18	ASM

System Name

Each storm drain subsystem should be assigned a unique System Name during model development. In HEC-RAS, the System Name defines a distinct pipe network, grouping all connected conduits and nodes under a single identifier and controlling how results and errors are reported. Using clear and unique System Names improves troubleshooting by allowing instability or error messages to be traced directly to a specific subsystem. To support efficient QA and debugging, system names should be assigned logically (e.g., by basin or tributary with a sequential ID) so problem areas can be quickly identified and isolated.

Modeling Approach

Use of the Instantaneous conduit option is recommended for short upstream collection laterals that convey flow to hydraulically modeled trunk lines, as it transfers flow directly between conduit ends without hydraulic routing. This reduces model complexity and runtime while still representing inlet behavior at nodes. Because hydraulic routing, storage, and backwater effects are not explicitly simulated, its use should be limited to upstream laterals feeding the hydraulically modeled system; consistent with this limitation, HEC-RAS only allows instantaneous conduits upstream of hydraulically modeled conduits. Mesh Cell Length

It is recommended to use a value on the order of pipe length divided by 3 as a starting point. A lower bound of about 1 foot and an upper bound of about 20 feet can be applied to maintain reasonable resolution. Additional refinement may be needed to support numerical stability. This is particularly important for steeper pipes, where finer mesh cells may be required. It is suggested to review initial conduit hydrographs, velocities, and Courant numbers to confirm stability and adjust the mesh as needed.

Pipe Shape and Manning’s “n”

Initial Assignments of Manning's roughness coefficient is based on pipe material type and cross-sectional shape using normal values as mentioned in Te Chow, 1959 and tabulated in Table 8 below. When pipe material is not provided or cannot be matched to a recognized code, a conservative fallback is applied using the Chow normal values for concrete; 0.013 for circular pipes and 0.015 for non-circular shapes.

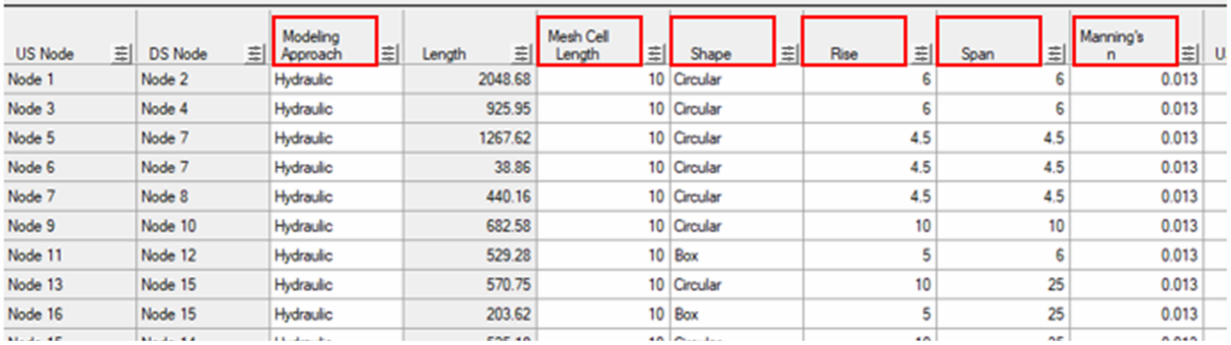
Table 8. Closed Conduits Mannings N Values

Material	Manning's N	Chow (1959) Reference	Alternative Codes
Concrete, circular	0.013	Concrete culvert with bends, connections, and some debris — normal	RCP, RCB, HERCP, RCP-CMP
Concrete, non-circular	0.015	Concrete sewer with manholes and inlets, straight — normal	RCP, RCB, HERCP, RCP-CMP
STEEL	0.024	Corrugated metal storm drain — normal	CMP, STL
HDPE	0.010	Brass smooth (closest smooth conduit analog) normal	CIPP, CIPP-CMP, PP, PVC
Clay	0.013	Common drainage tile — normal	—
Iron	0.013	Cast iron, coated — normal	DIP
Unknown	0.013 (circular); 0.015 (other)	Conservative default matching concrete guidance	Unmapped/missing materials

Minor Losses

Can use default HEC-RAS coefficients (US entrance = 0.2, DS exit = 0.4, US backflow = 0.4, DS backflow = 0.2 per RAS v7.0).

Table 9. Conduit Attribute Table with the Mandatory Fields to be Customized in Red Boxes



US Node	DS Node	Modeling Approach	Length	Mesh Cell Length	Shape	Rise	Span	Manning's n
Node 1	Node 2	Hydraulic	2048.68	10	Circular	6	6	0.013
Node 3	Node 4	Hydraulic	925.95	10	Circular	6	6	0.013
Node 5	Node 7	Hydraulic	1267.62	10	Circular	4.5	4.5	0.013
Node 6	Node 7	Hydraulic	38.86	10	Circular	4.5	4.5	0.013
Node 7	Node 8	Hydraulic	440.16	10	Circular	4.5	4.5	0.013
Node 9	Node 10	Hydraulic	682.58	10	Circular	10	10	0.013
Node 11	Node 12	Hydraulic	529.28	10	Box	5	6	0.013
Node 13	Node 15	Hydraulic	570.75	10	Circular	10	25	0.013
Node 16	Node 15	Hydraulic	203.62	10	Box	5	25	0.013

8.5.2.2 Nodes

Top/Side Inlet Parameters

HEC-RAS computes flow exchange between the surface and a pipe network node using rating curves generated from a drop (Top/Side) inlet weir length, a drop inlet orifice area, and corresponding coefficients. At shallow depths, weir flow controls capture; as ponding increases, the inlet transitions toward orifice control. This guidance explains how to compute the HEC-RAS weir length and orifice area inputs in a way that is consistent with HEC-RAS implementation while prioritizing Mile High Flood District: Urban Drainage and Flood Control District (UDFCD) inlet performance guidance.

For rectangular grate-type openings, HEC-RAS default inlet geometry is consistent with a three-sided effective weir (curb-side edge excluded) and an effective open area that is smaller than the full geometric opening. The relationships below in Table 10 reproduce the HEC-RAS default values and provide a clear geometric baseline for computing L_weir and A_orifice from inlet dimensions (W and L).

Table 10. Weir Lengths and Orifice Area Formulas Used in HECRAS Defaults

Input	HEC-RAS default relationship
Weir length, L_weir	L_weir = 2W + L
Orifice area, A_orifice	A_orifice = 0.8 × (W × L)

To confirm the baseline relationships above are aligned with HEC-RAS defaults, the computed values in Table 11 below match the two-standard grate-curb inlet defaults that HECRAS provides the values for:

Table 11. Verification of Default Formulas Used in HECRAS

Default inlet	W (ft)	L (ft)	Computed L_weir	Computed A_orifice	HEC-RAS default
2×3	2	3	7 ft	4.8 ft^2	7 ft / 4.8 ft^2
1.5×3	1.5	3	6 ft	3.6 ft^2	6 ft / 3.6 ft^2

While the baseline relationships replicate HEC-RAS defaults, MHFD/UDFCD has published inlet-specific laboratory-calibrated parameters for the inlet types that are most common in the District (Table 12). The UDFCD Technical Memorandum notes that unadjusted national methods can misrepresent these inlets and provides calibrated coefficients and reduction factors for sump (sag) conditions, including explicit factors applied to weir-length and orifice-area terms (sump equations) and recommended coefficients by inlet type. More details on these inlet types can be found in the UDFCD Technical Memo.

Table 12. Coefficients for Various Inlets in Sumps (Ref: UDFCD Memo)

Inlet type	Nw	Cw	No	Co
CDOT Type 13 Grate	0.70	3.30	0.43	0.60
Denver No. 16 Grate	0.73	3.60	0.31	0.60
Type 13 / No. 16 Combination – Curb Opening portion	1.00	3.70	1.00	0.66
CDOT Type R Curb Opening	1.00	3.60	1.00	0.67

Using MHFD/UDFCD reduction factors in HEC-RAS is appropriate because the HEC-RAS inlet rating curve is based on the same weir and orifice equations used in UDFCD guidance. In UDFCD formulations, the reduction factors (Nw and No) reduce the effective weir length and orifice area that control discharge capacity, which is equivalent to using adjusted (effective) geometry in the equations. This translates directly into HEC-RAS, since the same terms define the rating curve behavior.

A practical and recommended approach is to apply the UDFCD reduction factors directly to the geometric inputs in HEC-RAS. In other words, adjust the weir length and opening area by the reduction factors to represent their effective values, and use the corresponding UDFCD weir and orifice coefficients (Cw and Co) in the model. This keeps the implementation consistent with UDFCD methodology while aligning naturally with how HEC-RAS computes inlet hydraulics.

The procedures below in Table 13 describe how to compute HEC-RAS input values under two common data conditions. The first case applies when inlet type and dimensions are known and MHFD/UDFCD parameters can be used directly. The second case applies when inlet information is incomplete; in that case we still adopt MHFD/UDFCD assumptions by selecting a standard inlet size and a standard (documented) inlet type for parameterization.

Table 13. Example Calculations for L_{weir} and $A_{orifice}$ with Known and Unknown Inlet Datasets

Case	Inputs and assumptions	Compute HEC-RAS Inputs
A. Inlet type and opening dimensions (W, L) are known	Use the inlet-type-specific Nw, No, Cw, Co (and Cm as supporting context) from the UDFCD Technical Memorandum Table 1 for sump behavior.	Compute geometric terms: L_geom = 2W + L; A_geom = W×L. Then compute effective inputs: L_weir = Nw×L_geom; A_orifice = No×A_geom. Populate Cw and Co in HEC-RAS.
B. Inlet type and/or opening dimensions are unknown	Assume a standard 2×3 ft opening (W=2 ft, L=3 ft) and assume a standard MHFD grate type for parameterization. To avoid over-predicting capture when type is unknown, use the more restrictive of the common UDFCD grate parameters (Denver No. 16 Grate: Nw=0.73, No=0.31, Cw=3.60, Co=0.60) unless project-specific information indicates otherwise.	Use L_geom = 2W + L = 7 ft and A_geom = W×L = 6 ft^2. Then apply MHFD/UDFCD factors: L_weir = 0.73×7 = 5.11 ft; A_orifice = 0.31×6 = 1.86 ft^2. Populate Cw=3.60 and Co=0.60.

Inlets in Series

When communities provide a long ‘opening length’ (e.g., 120 ft), that value can be assumed to typically describe a repeated set of inlet units along a street and should not be entered directly as a single HEC-RAS weir length. Instead, as described in Table 14 below, represent the system as multiple inlet units distributed along the length, compute unit L_weir and A_orifice using Case A or Case B above, and then aggregate the total capacity by summing the unit inlet properties. Example values given a 120-foot total inlet opening size are presented in Table 15.

Table 14. Framework to Convert Long Opening Lengths into L_{weir} and $A_{orifice}$

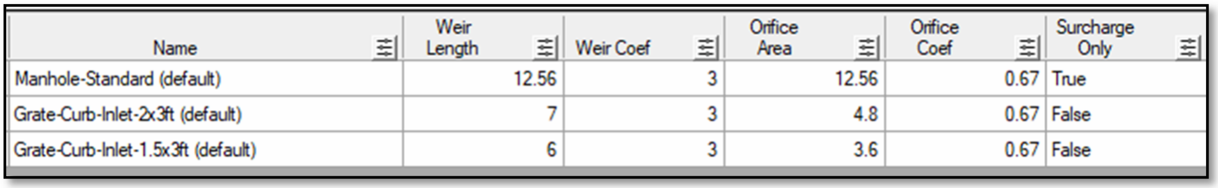
Item	Method
Equivalent number of inlets	N = L_opening / 10 (Perimeter length for a 2x3 Inlet)
Unit inlet parameters	Use Case A (known type) or Case B (unknown type) to compute L_weir,unit and A_orifice,unit
Aggregate inputs	L_weir,total = N×L_weir,unit; A_orifice,total = N×A_orifice,unit

Table 15. Example Calculation of L_{weir} and $A_{orifice}$ given a Known Opening Size

Example	L_opening	N	L_weir,total	A_orifice,total
Unknown type (Case B)	120 ft	12	61.32 ft	22.32 ft^2

The final calculated weir lengths and orifice area can be grouped into standard inlet type and sizes and a separate entry in the Top or side Inlet tables can be made for each group. Manholes should be set to “Surcharge Only = True.” This prevents the manhole from receiving inflow from the surface, while still allowing water to surcharge and discharge to the surface if system capacity is exceeded. These parameters are then used to generate a family of rating curves for the purpose of determining the flow exchange between the pipe network and the overlying surface water geometry where a drop inlet exists. For more details on the hydraulic equations being used, refer to the HECRAS Hydraulic Reference Manual - Flow at Drop Inlets section.

Table 16. Top/Side Nodes Attribute Table to be Customized



Name	Weir Length	Weir Coef	Orifice Area	Orifice Coef	Surcharge Only
Manhole-Standard (default)	12.56	3	12.56	0.67	True
Grate-Curb-Inlet-2x3ft (default)	7	3	4.8	0.67	False
Grate-Curb-Inlet-1.5x3ft (default)	6	3	3.6	0.67	False

Base Area

In HEC-RAS pipe networks, the Base Area in the Nodes table represents the manhole's cross-sectional area used to compute internal storage; using a non-zero value allows the node to store water and damp rapid water surface changes (improving stability and realism), while a value of zero removes storage and makes the node behave like a point with instantaneous response. Consistent with this, the City and County of Denver (2026 ICM Standards, Sec. 3.1.5, Unpublished as of May 2026) recommend setting the manhole area to match the physical structure (Chamber Plan Area), and where unknown, using a minimum of 12.56 ft^2 (4-ft diameter) to ensure a reasonable level of storage and avoid unrealistically sharp hydraulic responses.

8.5.3 Results Checks and Troubleshooting

Table 17 provides a focused set of QC checks for reviewing storm sewer pipe results in HEC-RAS 2D pipe network models. Each row links a common result pattern to the most relevant diagnostic check and the preferred corrective action, helping isolate whether an issue is driven by numerical stability, pipe–surface coupling, inlet definition, or geometry setup. The goal is to achieve stable conduit hydrographs throughout (Figure 32).

Table 17. Recommended Checks and Fixes for Common Pipe Result Errors

QC check (issue + what to look for)	Recommended fix / adjustment	Why it matters
Check for pipe–2D coupling lag near culvert openings or exchange-heavy nodes.	Turn on "Compute Every 2D Iteration" for the pipe system (expect longer runtime). (Figure 33)	Reduces iteration lag between updated boundary heads and pipe computations, improving stability/accuracy.
Check early timesteps of conduit flow and node Hydraulic Grade Line (HGL) for a startup surge.	Use pipe-network initialization options (project initial WSE from downstream and/or ramp from upstream with warm-up). (Figure 33)	Startup artifacts can invalidate peak HGL and confuse stability diagnostics.
Check high-HGL nodes for missing Top/Side inlet or incorrect Surcharge Only setup.	Define a manhole at nodes where surcharge must be allowed to leave the node. Use "Surcharge Only" to block inflow while still allowing surcharge out.	Trapped head can push unrealistic pressures upstream and shift where the system appears to fail.
Check inlet weir/orifice parameters and coefficients for unrealistic exchange.	Adjust inlet geometry/coefficients to reasonable values and rerun. (Table 16)	Exchange settings govern how the network is loaded and relieved, directly affecting flows and HGL.

QC check (issue + what to look for)	Recommended fix / adjustment	Why it matters
Check upstream vs downstream mesh cell face flow in the hydrograph viewer for unintended reverse flow.	Verify conduit's digitized direction, inverts, and slope; also check downstream submergence/stage control.	Reverse flow can indicate a geometry direction issue or backwater control, changing bottleneck interpretation.
Check Pipe Velocity and Courant for outliers along profiles.	Verify pipe size/shape and inverts first; then adjust roughness if needed and rerun. (Table 9)	Extreme Velocities (<4 fps or >18fps)and high Courants (>1) are fast reasonableness checks for geometry and conveyance, which often indicate bad inputs.

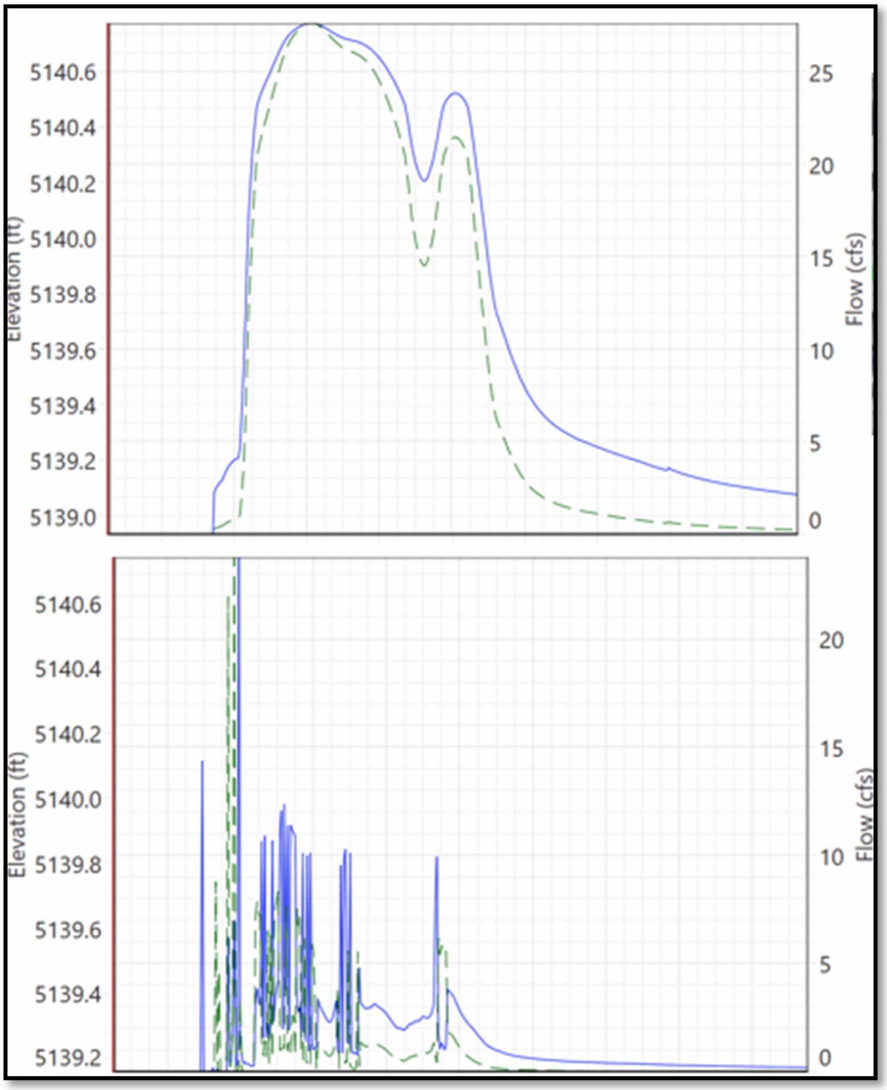
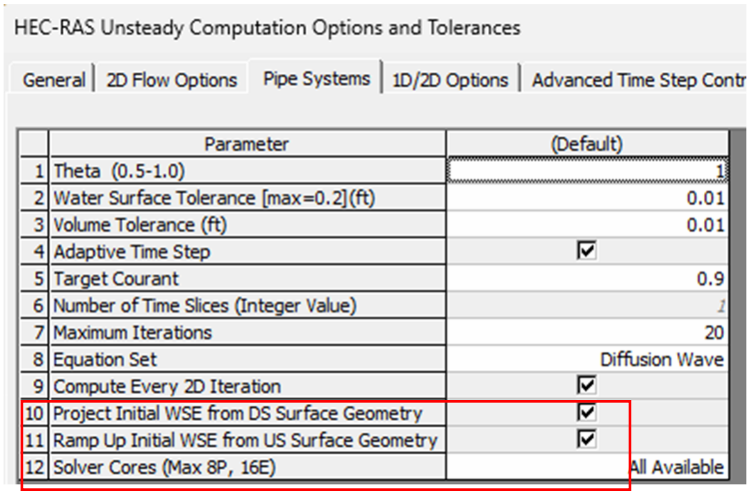


Figure 32. Pipe Flow Hydrograph Example, Stable (Top) and Unstable (Bottom)



	Parameter	(Default)
1	Theta (0.5-1.0)	1
2	Water Surface Tolerance [max=0.2](ft)	0.01
3	Volume Tolerance (ft)	0.01
4	Adaptive Time Step	☑
5	Target Courant	0.9
6	Number of Time Slices (Integer Value)	1
7	Maximum Iterations	20
8	Equation Set	Diffusion Wave
9	Compute Every 2D Iteration	☑
10	Project Initial WSE from DS Surface Geometry	☑
11	Ramp Up Initial WSE from US Surface Geometry	☑
12	Solver Cores (Max 8P, 16E)	All Available

Figure 33. HECRAS Computation Settings for Pipe Systems

9 Simulation Solution Equations and Parameters

The choice of solution equations and computational parameters influences the numerical behavior of the hydraulic model. This chapter discusses recommended computation settings, advanced timestep control, and model stability criteria. Standardizing these parameters provides consistency so that simulations are reproducible, errors are minimized, and results are comparable across studies.

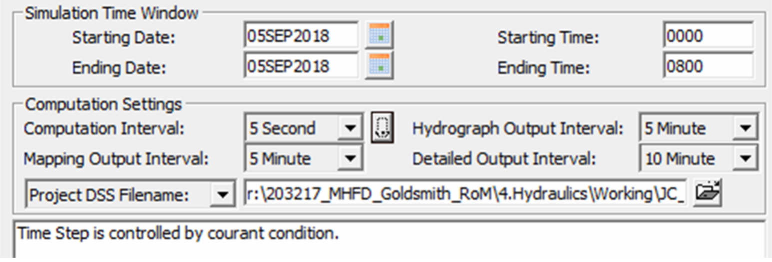
9.1 Computation Settings

9.1.1 2D Flow Area

Computation, hydrograph output, mapping output and detailed output intervals may vary depending on domain size and complexity. The recommended settings are shown below in Table 18 and Figure 36. Computation intervals can be decreased or increased to improve run time and model stability as long as both metrics are satisfactory. Using adaptive timesteps based on courant values is also required for improved stability. Note the mapping output interval should be refined (5 to 30 minutes) for the key design storm events: 5-, 10-, 100-, and 500-year. For all other design storms, it is acceptable to increase the mapping output interval from 30 minutes to 1 hour. In rare cases where even with 1-hour mapping output intervals result in .hdf files that are too large, the max profile may be used; however, the decision process should be well documented in the accompanying report.

Table 18. Computation Settings

Setting	Time Interval	Description	Potential Considerations
Computation Interval	**1 to 30 seconds** with adaptive timestep controls	The computation interval is the timestep used to solve the governing hydraulic equations.	Timesteps that are too large may result in instabilities, specifically around complex structures. Timesteps that are too small will inflate run times, so it is essential to select a balanced value with adaptive controls based upon courant values.
Hydrograph Output Interval	**1 to 15 minutes**	The hydrograph output interval is used to define the stage and flow hydrograph intervals that are written to the HEC-DSS file	Stored in DSS – generally small
Mapping Output Interval	**5 to 30 minutes** for 5-, 10-, 100-, and 500-year events **30 minutes to 1 hour** for all other design storm events	The mapping output interval is used to determine the interval at which the resulting maps will be viewable in RAS Mapper.	Mapping output intervals generally have a large impact on file size. If the mapping output interval is too small it can greatly increase *.p.hdf file sizes. Longer (less frequent) mapping output intervals should be used for non-primary events.
Detailed Output Interval	**10 to 30 minutes**	Is used to write out profiles of WSE and flow.	Detailed output intervals can have a large impact on file size. It is recommended to keep the detailed output interval as large as possible within reason to reduce overall file size.



Simulation Time Window
Starting Date: 05SEP2018
Ending Date: 05SEP2018
Starting Time: 0000
Ending Time: 0800

Computation Settings
Computation Interval: 5 Second
Mapping Output Interval: 5 Minute
Hydrograph Output Interval: 5 Minute
Detailed Output Interval: 10 Minute
Project DSS Filename: r:\203217_MHFD_Goldsmith_RoM\4.Hydraulics\Working\JC_

Time Step is controlled by courant condition.

Figure 34. Example Unsteady Flow Computation Settings

9.1.2 Pipe Systems

9.1.2.1 Grouping Pipe Systems

The System Name groups pipes into separate subsystems that are solved independently, and it is also used directly in HEC-RAS error messages. Assigning clear and distinct System Names early in model development is recommended, as it makes it much easier to identify which part of the network is causing instability and focus troubleshooting in large models.

At the same time, there is a balance to maintain with performance. While separating systems improves troubleshooting, having too many small subsystems can reduce computational efficiency. HEC-RAS processes pipe systems in parallel, but excessive fragmentation can introduce overhead and slow the model down. As the model stabilizes and major issues are resolved, it is generally recommended to consolidate smaller systems into a smaller number of larger subsystems to improve runtime and better utilize available processing power.

As a practical guideline, aim to keep the total number of pipe systems and 2D flow areas combined at or below the number of computation cores assigned to the simulation. This helps ensure the model can take full advantage of parallel processing without unnecessary overhead. Overall, the goal is to start with more systems to support efficient troubleshooting, then simplify the setup once the model is stable to improve performance.

9.1.2.2 Manual vs Courant Based Time Steps

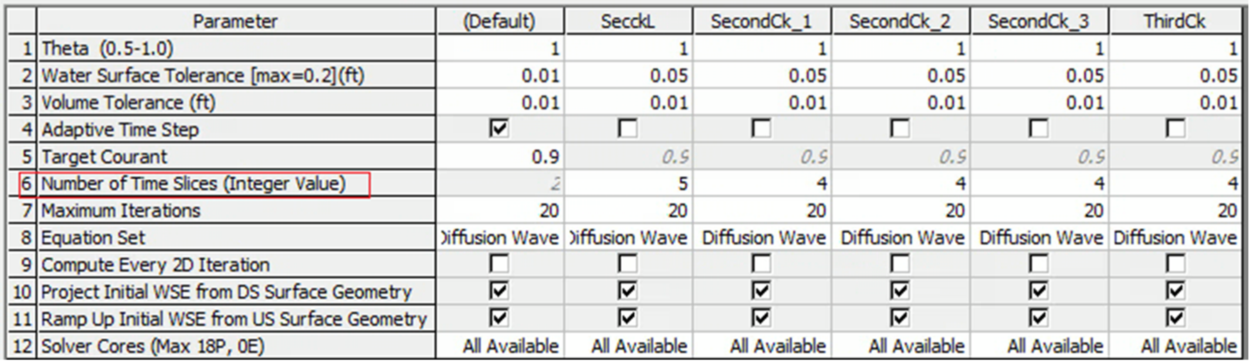
The pipe network in HEC-RAS 7.0 computes by default using an internal time step that is governed by an adaptive Courant-based approach, typically targeting a value around 0.9. In systems with short conduits and high velocities, this can result in very small pipe time steps, even when the overall model remains stable. This behavior can significantly increase runtime without a corresponding gain in accuracy, because the semi-implicit pipe solver remains stable at larger time steps and is not strictly constrained by Courant limitations.

A more practical approach for coupled 2D–pipe models is to disable adaptive time stepping for the pipe network and instead use a fixed time step consistent with the 2D surface, combined with a defined number of pipe sub-steps (“Time Slices”). The 2D Flow Area time step should always be set based on Courant stability, and the pipe system should then be sub-stepped within that interval. For example, if the 2D time step is 1.0 second, using 5 pipe time slices results in pipe time steps of 0.2 seconds, which is typically sufficient for stability without over-resolving the solution. In most cases, starting with 3 to 5 pipe time slices per 2D time step is appropriate, with adjustment only if instability is observed, whether in Pipe WSE convergence errors or high oscillations in pipe conduit hydrographs.

If pipe instability errors occur during a simulation run, such as “Pipe Solution went unstable,” resulting in the termination of the plan run, first increase the number of pipe time slices up to about 10 to further refine the internal solution for that specific pipe system. If instability still occurs, particularly in large grouped subsystems where isolating and correcting specific problem pipes through targeted GIS edits is difficult, it is acceptable to re enable Courant based adaptive time stepping for only those affected subsystems. This should be used as a focused fallback to improve numerical stability when direct troubleshooting is impractical, rather than as the default approach for the entire model.

9.1.2.3 Compute Every 2D Iteration

This optional setting forces a pipe network system to be re-solved during each 2D area iteration (sub-iteration) within a time step, rather than only once per step. By default it is off, meaning the pipe network is updated *after* the 2D area converges each time step. When enabled, it can improve stability for tightly coupled flows (e.g., pipes heavily exchanging with a 2D area via culvert openings) by updating pipe results immediately as 2D water levels change. However, this comes at a steep performance cost: a large pipe network will be solved many times per interval, drastically increasing runtime. The recommended practice is to leave this option OFF, using it only in specific scenarios, such as at persistent 1D–2D convergence issues.



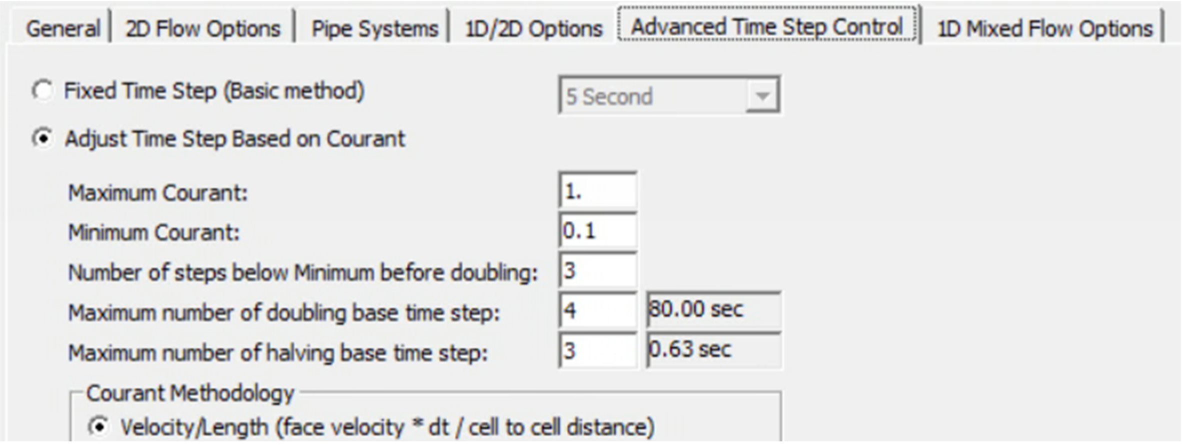
	Parameter	(Default)	SeckL	SecondCk_1	SecondCk_2	SecondCk_3	ThirdCk
1	Theta (0.5-1.0)	1	1	1	1	1	1
2	Water Surface Tolerance [max=0.2](ft)	0.01	0.05	0.05	0.05	0.05	0.05
3	Volume Tolerance (ft)	0.01	0.01	0.01	0.01	0.01	0.01
4	Adaptive Time Step	☑	☐	☐	☐	☐	☐
5	Target Courant	0.9	0.9	0.9	0.9	0.9	0.9
6	Number of Time Slices (Integer Value)	2	5	4	4	4	4
7	Maximum Iterations	20	20	20	20	20	20
8	Equation Set	)iffusion Wave	)iffusion Wave	Diffusion Wave	Diffusion Wave	Diffusion Wave	Diffusion Wave
9	Compute Every 2D Iteration	☐	☐	☐	☐	☐	☐
10	Project Initial WSE from DS Surface Geometry	☑	☑	☑	☑	☑	☑
11	Ramp Up Initial WSE from US Surface Geometry	☑	☑	☑	☑	☑	☑
12	Solver Cores (Max 18P, 0E)	All Available	All Available	All Available	All Available	All Available	All Available

Figure 35. Example Computation Settings for Pipes with Grouped Systems and Manual Time Slices

9.2 Solution Equations and Advanced Timestep Control

Start 2D HEC-RAS model development with the Diffusion Wave (DW) solver to improve runtime and stability, especially for large urban systems with many hydraulic structures. The DW approach is less sensitive to rapid momentum changes, so it can run with larger time steps and higher Courant numbers (up to about 3), which helps speed up early testing and troubleshooting. Use Advanced Time Step Control so the model can automatically reduce the time step as needed and set a small minimum time step (for example, 0.5 seconds) as a safety limit during rapidly changing conditions. The minimum time step in the advanced controls is simply the smallest step the model will take when automatically halving for stability and is not the same as the regular base computation interval you set for routine calculations (Figure 36).

After the model is stable, evaluate whether full momentum is needed using a focused comparison and site conditions. Create a Shallow Water Equations (SWE) plan and compare results at key locations such as bridges, culverts, storage nodes, and critical flood elevations. Switching to SWE is recommended in low-slope systems where inertia effects are more important, generally on the order of 1 foot per mile or flatter, as well as in steep urban systems where rapid acceleration, flow transitions, or hydraulic jumps are expected (Shallow Water or Diffusion Wave Equations). If the SWE results show meaningful differences, such as higher water surface elevations, different flow distribution, or changes in overtopping or flood extent, then use the SWE solver for final calibration and mapping. If differences are minor, for example <0.5 foot, in stage and no significant change in mapped extents, the DW solution may be retained for final use with proper documentation. This approach keeps the model efficient while still applying full momentum where it meaningfully improves results.



○ Fixed Time Step (Basic method) 5 Second

● Adjust Time Step Based on Courant

Maximum Courant: 1.

Minimum Courant: 0.1

Number of steps below Minimum before doubling: 3

Maximum number of doubling base time step: 4 80.00 sec

Maximum number of halving base time step: 3 0.63 sec

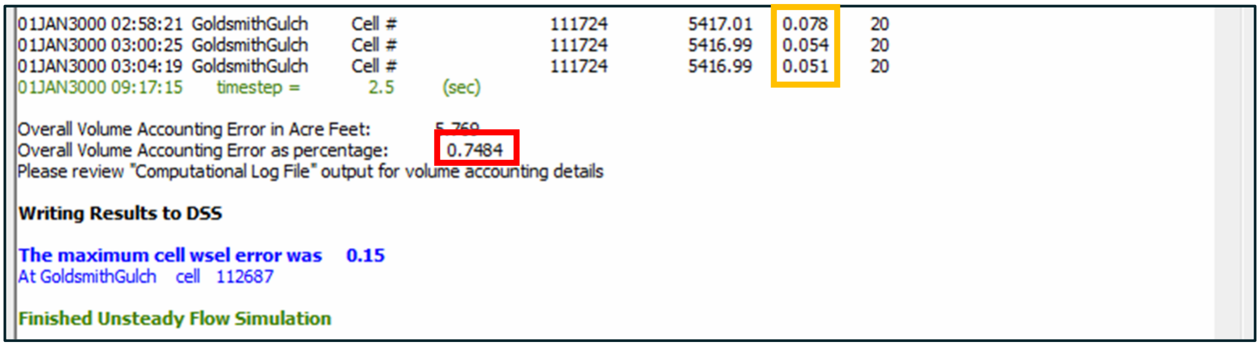
Courant Methodology

● Velocity/Length (face velocity * dt / cell to cell distance)

Figure 36. Example Advanced Timestep Control Settings

9.3 Model Stability and Errors

Model stability is essential for defensible results. Total volume accounting errors should not exceed 2%. No individual cell WSE error may exceed 0.2 foot along the main channel without reasonable documentation and justification. All computation output logs should be visible and run without errors. Any large volume and WSE errors should be thoroughly troubleshooted and then documented and explained if unresolved. Figure 37 is an example of the stability records in the computation output log.



```
01JAN3000 02:58:21  GoldsmithGulch     Cell #              111724        5417.01   0.078     20
01JAN3000 03:00:25  GoldsmithGulch     Cell #              111724        5416.99   0.054     20
01JAN3000 03:04:19  GoldsmithGulch     Cell #              111724        5416.99   0.051     20
01JAN3000 09:17:15      timestep =        2.5        (sec)

Overall Volume Accounting Error in Acre Feet:        5.769
Overall Volume Accounting Error as percentage:       0.7484
Please review "Computational Log File" output for volume accounting details

Writing Results to DSS

The maximum cell wsel error was     0.15
At GoldsmithGulch    cell    112687

Finished Unsteady Flow Simulation
```

Figure 37. Example Volume Accounting Error (red box) and Maximum Cell WSE Error (orange box)

9.4 Potential Model Run Time Improvements

Runtimes can often be reduced without sacrificing solution quality by tuning the computation interval and using variable time-step controls that respond to Courant conditions to avoid unnecessarily small steps that increase runtime yet still fail to resolve local instabilities. In practice, efficient models are obtained by selecting the largest stable computation interval, limiting the number of times the base time step is halved to what is needed, and then correcting instability at the source rather than globally over-restricting the simulation. For 2D domains, this means identifying cells that produce persistent water surface elevation errors and resolving the underlying problem where possible, such as terrain artifacts, poor local connectivity, or excessive cell distortion. Local fixes are generally more effective than blanket timestep reduction after initial results are obtained.

10 Calibration

Calibration is the process of aligning model outputs with observed data so that the model better represents reality and produces more accurate and reliable results. This chapter presents methods and resources for calibration and validation, including the use of historical events, gage analysis, and statistical performance metrics. Consistent documentation and parameter adjustment protocols are essential for reproducibility and regulatory compliance. Standardized calibration practices enable modelers to demonstrate agreement with observed flood behavior, justify parameter choices, and produce models that support sound engineering decisions.

10.1 Methods and Resources for Calibration/Validation

To achieve reliable calibration and validation, prioritize data sources and methods that allow the most comprehensive representation of watershed response. Selection should be based on data availability and event characteristics. Table 19 contains calibration methods and data types ranked in terms of preference for the Risk Framework program and the calibration type required.

Calibration refers to adjusting watershed response parameters (e.g., Curve Numbers, Manning’s n) to better match modeled results with observed data. Validation involves applying those calibrated parameters to an independent dataset (typically of lower confidence) and limiting adjustments to event-specific inputs such as initial conditions (baseflow, reservoir levels) or timing shifts, without re-tuning core parameters. Events classified under calibration should always be prioritized, with validation methods serving as supplemental support.

Table 19. Calibration Methods and Implementations

Order	Method	Validation or Calibration	Details (Preferred Use & Implementation)	Calibration Type
1A	District GARR	Calibration	Apply locally developed Gage Adjusted Radar Rainfall (GARR factors to adjust point rainfall to basin-average values for historical events. Ensures spatial consistency and aligns with District standards. GARR provides data at 5-minute resolution dating back to 2012.	Full hydrograph
1B	MRMS / AORC	Calibration	Use gridded precipitation datasets for selected historical events: Multi-Radar/Multi-Sensor (MRMS) for events since 2014 and Analysis of Record for Calibration (AORC) for events between 1979 and 2014. Import grids into the model and synchronize with event timing. Both provide 1-kilometer resolution, with MRMS at 2-minute intervals and AORC at hourly intervals.	Full hydrograph
1A/B	Stream Gage	Calibration	Continuous or peak stage and flow hydrographs for historical events (Figure 38). Commonly available stream gages within the District include ALERT and USGS gages. Precipitation data for stream gage calibration should come from GARR, MRMS, and AORC data.	Full hydrograph

Order	Method	Validation or Calibration	Details (Preferred Use & Implementation)	Calibration Type
2	LPIII Gage Analysis FFA	Validation	Perform frequency analysis using gage records to derive flow-frequency curves. Compare simulated peaks to expected values and adjust parameters if outside confidence limits. Precipitation data for LPIII gage analysis comes from NOAA Atlas 14.	Peak flows (±1 SD)
3	Historic Imagery	Calibration	Historical imagery or high water marks georeferenced and time stamped. These can be simulated with GARR, AORC, or MRMS precipitation.	Stage/Extent only
4	Unit Discharges	Validation	Where no above calibration methods are available, unit discharges can be compared between adjacent watersheds of similar characteristics. Watersheds should be adjacent with similar percentages of land use, soil type, area, and channel geometry. Discharges should be area adjusted based on total drainage area.	Peak flows
5	Prior Detailed Studies	Validation	Comparing RoM-derived flows against prior detailed studies. Greater weight should be given to prior models that were calibrated, and ideally the RoM results should be compared against the same calibration source data, rather than the model itself. Caution should be taken when comparing results from one model (RoM) to another (prior FHAD) and should only be done as a reasonableness check.	Peak flows and inundation extent
6	Regional Regression Equations	Validation	Estimate peak flows using published regression equations when gage data are limited. Validate modeled peaks against computed flows for similar recurrence intervals. There is a limitation in RREs in that they are often not trustworthy in large urban areas or watersheds with significant retention and storage. This may make them marginal for most of the MHFD area.	Peak flows (±1 SD)

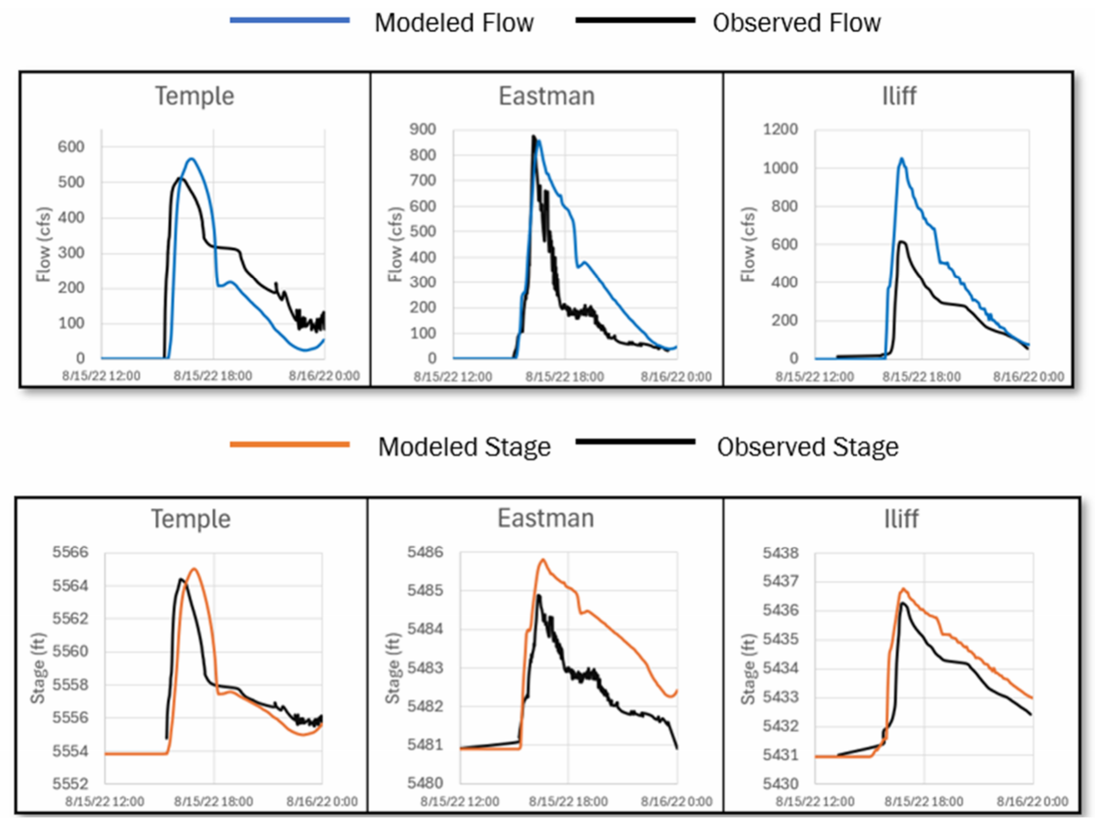


Figure 38. Flow and Stage Hydrographs at Calibration Gages (upstream to downstream)

10.2 Historical Calibration (District GARR / MRMS /AORC)

Where possible, the full profile of a historical flow and stage hydrograph should be used for a volumetric calibration, rather than focusing only on peak flow (SID 59). This approach evaluates the full flow profile, including rising and recession limbs, which is essential for capturing overland flow paths, time of concentration, and runoff volume. Furthermore, volumetric calibration becomes critical if the gage or observation point is located within the influence of a storage facility, as outflow rating curves or outlet structures become highly influential. The following parameters should be adjusted in the listed order of priority.

Curve Numbers: Can be adjusted within the model to alter total volume and therefore also peak flow.

Channel Manning's "n": Channel roughness values can be adjusted to alter peak timing and stage elevations.

Shallow and Overland Manning's "n": Can be adjusted for timing and flow hydrograph shape.

Initial Conditions: Initial conditions points for reservoir elevation, baseflows, storage operations (if justified), etc., can be adjusted to calibrate historical events and design storms if it does not disagree with known historical information.

Calibration should be performed using two storm events to capture a representative range of watershed behavior. Reliance on a single event introduces a high risk of overfitting model parameters to one historical condition and can bias the resulting design storm response, while calibrating to a large number of events is generally not practical. Using a smaller, more frequent event (e.g., 20% to 10%) alongside a larger, less frequent event (e.g., 2%) provides an efficient and balanced approach. While larger infrequent events have traditionally been emphasized in FEMA studies (e.g., 1%) due to regulatory implications, smaller and more frequent events should be prioritized for Flood Risk applications because they carry greater overall influence on flood risk scores. Where data allows, a third midsize event (e.g., 4%) could be used as a validation check to confirm that parameter adjustments remain balanced. In all cases, the feasibility of this approach depends on the available gage record within the study area, but the intent is to maintain a practical, multi-event calibration that remains representative of actual watershed conditions.

The entire calibration process, including input data, parameter changes, rationale, and results, should be documented in detail (SID 61). This documentation supports reproducibility and compliance with FEMA's requirements. Calibration must also ensure agreement between modeled results and regulatory products, applying sound engineering judgment throughout (SID 62). Model performance statistics are used to quantitatively assess calibration performance across the watershed. Table 20 provides the industry-standard performance ratings (D. N. Moriasi et al., 2007, 2015).

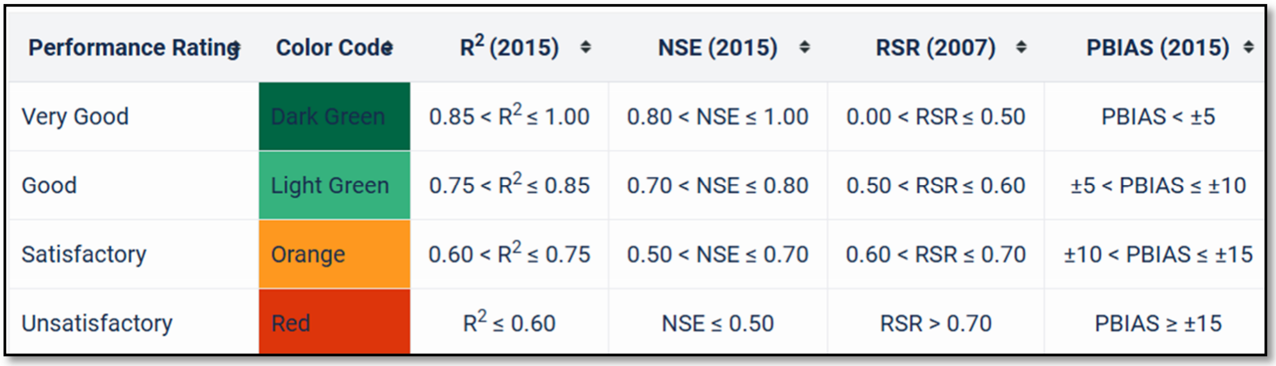
Frequently, conditions that differ from the design events must be assumed and represented in the model to replicate historic events. The objective for calibration events is to reflect historic conditions, while the objective for design events is often to make conservative assumptions that may provide a slightly higher factor of safety. Differences typically apply to initial conditions concerning storage and starting WSE in reservoirs and detention ponds. Calibration events for these areas use gage data or historic records when available, as opposed to design events that use the normal pool elevation (if provided) or a higher WSE providing less storage/flow mitigation. This also pertains to conveyance assumptions for regulated features such as canals and ditches, which are typically available for full or partial flow in calibration events, but blocked (consumed storage) for design events. Occasionally, there is reason to assume different initial abstraction conditions during calibration for unique circumstances, such as prolonged drought (drier soils) or prior rainfall with wetter conditions (e.g., the 2013 event).

10.3 Calibration Performance Metrics

Historical calibration should be evaluated with the metrics outlined in Table 20.

- Nash-Sutcliffe Efficiency (NSE): Represents the variance of the model's errors compared to the variance of the observed data. An NSE of 1 indicates a perfect model fit.
- Coefficient of Determination (R^2): Determines how well-observed outcomes are replicated by the model, or if there is a correlation between the two datasets.
- Root Mean Standard Deviation Ratio: Is the average difference between the simulation results and the observed historical data normalized by the standard deviation of the observed data, or Root mean square error divided by standard deviation.

Table 20. Calibration Performance Ratings for Full Runoff Hydrographs (Moriasi et al., 2007, 2015)



Performance Rating	**Color Code**	**R^2 (2015)**	**NSE (2015)**	**RSR (2007)**	**PBIAS (2015)**
Very Good	Dark Green	0.85 < R^2 ≤ 1.00	0.80 < NSE ≤ 1.00	0.00 < RSR ≤ 0.50	PBIAS < ±5
Good	Light Green	0.75 < R^2 ≤ 0.85	0.70 < NSE ≤ 0.80	0.50 < RSR ≤ 0.60	±5 < PBIAS ≤ ±10
Satisfactory	Orange	0.60 < R^2 ≤ 0.75	0.50 < NSE ≤ 0.70	0.60 < RSR ≤ 0.70	±10 < PBIAS ≤ ±15
Unsatisfactory	Red	R^2 ≤ 0.60	NSE ≤ 0.50	RSR > 0.70	PBIAS ≥ ±15

10.4 Alternative Calibration Options

When historical event data are unavailable, first determine whether adjacent watersheds have already been calibrated under the MHFD's Flood Risk program. If they have and share similar mesh characteristics, import calibrated parameters such as CNs and Manning's "n" assignments as base values, verifying that land cover classes match between the watersheds. Nearby gaged watersheds can also be consulted for unit discharges (cfs/sq. mi.) if they share similar characteristics and flow dynamics, which will not often be the case in developed areas. These sources should primarily be used for validation and reasonableness checks rather than adjusting calibration parameters in the absence of observed data. In other words, these sources should not be used as targets to match, but as ranges to check that results are in the right ballpark.

If parameters are adopted from nearby watersheds, they should be validated to the extent possible against gage-informed LPIII analysis, historic imagery, and/or regression equations (last resort). The primary instance in which an LPIII analysis would be used for statistical forcing (e.g., 10% rainfall ≈ 10% flow) is when the gage record has more than 10 years, the record predates the AORC (before 1979), and the watershed has not undergone major changes since then. While encountered frequently in remote undeveloped watersheds, this seldom occurs (if ever) in the District.

Modeled flows should generally fall within one standard deviation of the regression estimates, but modeling parameters should not be adjusted to closely match regression estimates that can often be fairly inaccurate. Regression equations should be reserved as a "validation" check within a tolerance range for less developed domains (e.g., near the foothills or near Denver International Airport) but should not be consulted for urbanized areas with stormwater infrastructure and detention.

Another valuable source for calibration is high water marks and pictures from historic events. While it's preferred to have surveyed high-water marks, even field measurements, news articles (e.g., a depth reported at an intersection), or a picture of debris post-flood are all useful datasets. Timesteps are important context, and if unknown, the data still provide a floor WSE knowing that the flood was at least that deep. Common features present in the picture can help estimate approximate depths (e.g., people, cars, fire hydrants, street signs, etc.) and pictures can often be located by distinct features or signage (e.g., street signs, store fronts, pathways, addresses, etc.).

11 Creating Pseudo-Steady State Riverine Models

The steps taken thus far and described in the above sections arrive at a fully developed, calibrated 2D RoM model. The RoM model will be used to generate pluvial (overland ponding) hazard and risk information, which will remain a non-regulatory, locally referenced dataset. Fluvial (riverine) hazard and risk information will be generated from a pseudo-steady state (PSS) riverine model derived from the RoM model following the steps described in this section.

11.1 Purpose

Having a PSS riverine model comes with several advantages and is far more compatible with today's regulatory framework, thus allowing riverine products to be considered for regulatory use. This derivative model enables the Flood Risk Program to remain flexible and focused on its primary role as a planning and capital prioritization tool, while advancing potential regulatory updates in the future with relatively minimal additional effort.

The primary purpose of the PSS approach is to produce riverine flow conditions that are consistent with regulatory expectations while maintaining the hydraulic fidelity of the underlying 2D framework. The calibrated RoM model captures realistic watershed behavior, including storage and attenuation from terrain and infrastructure; however, these effects can conflict with regulatory requirements where non-engineered storage is not permitted to reduce downstream flows. The PSS approach addresses this by removing unintended attenuation and generating flow conditions that are more stable, transferable, and suitable for regulatory applications such as floodway analysis, no-rise evaluations, and future map revisions.

11.2 Methodology

The following approach will be used to effectively decouple fluvial hydrology and hydraulics. While this adds a layer of complexity to data development, it considerably simplifies the potential regulatory use and maintenance of riverine models. This decoupling ensures that hydrologic inputs are preserved independent of changes to hydraulic routing or storage effects, allowing flows to be consistently applied across scenarios without being influenced by localized attenuation or backwater effects observed in the RoM simulation.

Note: The following workflow is provided as a recommended methodology for application in future studies and should be adapted as needed based on project-specific conditions and evolving District guidance.

11.2.1 Develop a Structures-Removed Reference Condition

Create a copy of the final calibrated RoM geometry and modify it to represent a fully drained, non-attenuated condition.

All hydraulic structures should be removed from the geometry, including culverts, bridges, stormwater connections, and detention outlets. Any residual terrain artifacts that may continue to impound flow should be smoothed to ensure continuous downstream conveyance. The objective is to eliminate both engineered and inadvertent storage effects so that flow propagates without attenuation.

Reference lines should be added throughout the system at key hydraulic control locations, as illustrated in Figure 39. These locations typically include upstream and downstream of major crossings, confluences, and any locations where flow additions or losses are expected. The spacing of reference lines should be sufficient to capture changes in flow along the reach.

The model should then be simulated for all required recurrence intervals using this modified geometry, with a 50-year restart file developed in the same manner as for the final RoM design simulations (note that the restart file will need to be produced again following changes to the geometry). The resulting hydrographs will represent the baseline condition of unattenuated flow through the system.

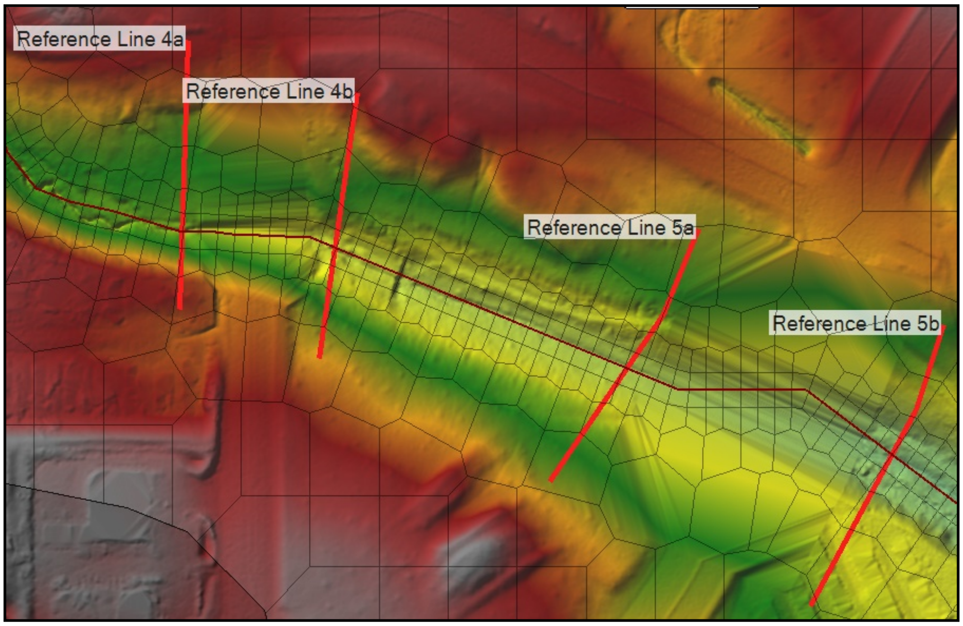


Figure 39. RoM Model Geometry with Structures Removed and Reference Lines Added (in red)

11.2.2 Extract Target Flows and Compute Incremental Inflows

From the structures-removed simulation, extract peak flows at each reference line for all recurrence intervals. These peak flows represent the non-attenuated target condition that the PSS model must reproduce. Incremental inflows are then computed as the difference in peak flow between consecutive reference lines moving downstream.

For example, consider four reference lines along a reach with the following extracted peak flows for a given recurrence interval:

Ref Line	Station (ft)	Peak Flow (cfs)	Incremental Flow (cfs)
RL-1	0	500	500 *(upstream boundary)*
RL-2	2,000	800	300
RL-3	5,000	1,100	300
RL-4	8,000	1,300	200

In this example, the upstream boundary condition at RL-1 is set equal to the extracted peak flow (500 cfs). The incremental inflow at each downstream reference line is then computed as the difference between adjacent peak flows. For instance, the incremental inflow at RL-2 is 800 − 500 = 300 cfs, and similarly for downstream locations.

When applied in the PSS model, these incremental flows are assigned as constant values via lateral inflow boundary conditions at their respective reference lines. The cumulative effect of these inflows reconstructs the target flow profile, such that the modeled flow at each location matches the peak flows extracted from the structures-removed simulation. For example, flow at RL-3 becomes 500 + 300 + 300 = 1,100 cfs, confirming consistency with the target condition.

This process should be repeated for all recurrence intervals using a consistent set of reference lines. Results should be organized in a spreadsheet with peak flows and incremental differences clearly tabulated, and a simple cumulative check should be performed to confirm that the reconstructed flows match the extracted target flows at each location.

11.2.3 Construct the PSS Geometry and Apply Distributed Inflows

Create a new geometry by copying the final calibrated RoM geometry. Unlike the structures-removed condition, this geometry should retain hydraulic structures and terrain features needed for realistic hydraulic behavior.

At each reference line, apply lateral inflow boundary conditions corresponding to the incremental flows computed in the previous step. These inflows are applied sequentially moving downstream so that the cumulative flow at each location matches the target peak flow extracted from the structures-removed condition. These inflows are implemented as constant or quasi-steady peak flow values rather than full hydrographs. This removes the influence of timing and storage effects and approximates a steady-state condition within the 2D framework.

The model is then simulated for all recurrence intervals using these distributed inflow conditions, without applying any initial condition restart file, as the influence of antecedent conditions is already incorporated in the target flows derived from the structures-removed simulation.

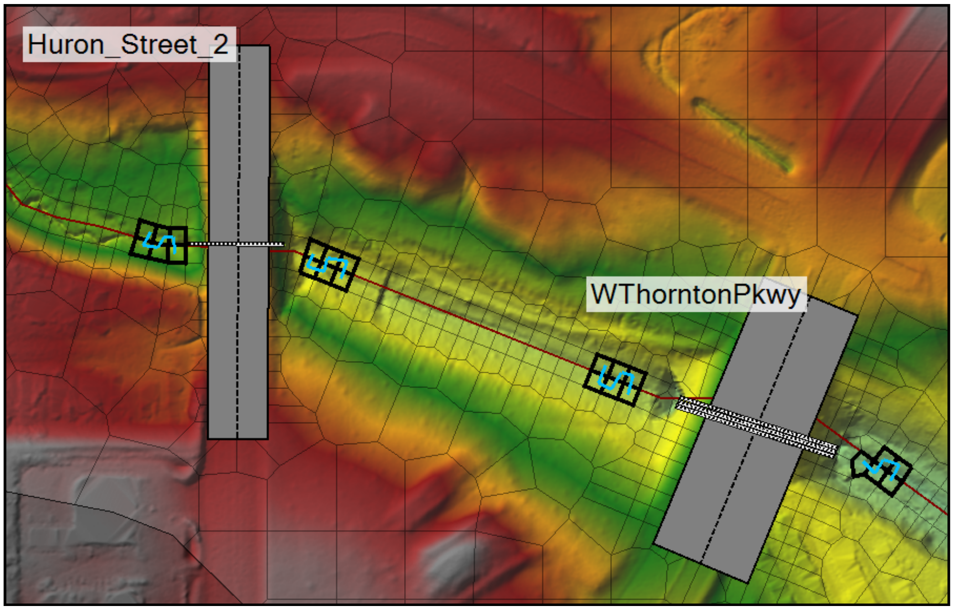


Figure 40. PSS Model Geometry with Lateral Inflows near Structures

11.2.4 Update the Land Cover for Approximated Depth-Weighted Roughness

Once the PSS model has established reasonable flows, the land cover layer ought to be updated from the version created for the RoM model (Section 5.1.4) to account for different flow regimes – or “depth corridors” in the PSS model. For the RoM model, a single depth-weighted approximation is reflected in the land cover layer shared across all 13 recurrence intervals, which is reasonable as the vast majority of the domain falls under the “overland sheet flow” category. However, the PSS model is exclusively focused on the riverine corridor, whose flow regime changes considerably between the 1-year and the 1000-year event. For this reason, two land cover layers will be generated: one generated with the 5-year event (LandCover_PSS_small) for use in the 1 to 25-year events, and one generated with the 100-year event (LandCover_PSS_large) for use in the 50 to 1000-year events.

11.2.5 Iterative Calibration to Match Target Flow Conditions

Compare the results of the PSS simulation against the target flows derived from the structures-removed condition. At each reference line, verify that modeled peak flows match the target peak flows within an acceptable tolerance (±10%). Water surface elevations and profiles should also be reviewed for consistency and stability.

If discrepancies are observed, adjustments should be made to the magnitude or distribution of lateral inflows. In some cases, refinement of reference line locations or redistribution of incremental flows may be required. Simulation time and boundary condition duration may also need adjustment to ensure stable peak conditions are achieved. This process should be repeated iteratively until the PSS model reproduces the target flow profile throughout the system while maintaining stable hydraulic results.

11.2.6 Finalization and Documentation

Once convergence is achieved, the PSS model represents the regulatory riverine condition for the study. Results should be exported from the final timestep, not the maximum across all timesteps. All reference line locations, extracted flows, incremental calculations, and applied boundary conditions should be documented in a reproducible format. This documentation is critical for future model updates, regulatory review, and consistency across studies.

12 Model Outputs and Post-Processing

Once the models have been calibrated, they will be run for the 13 design storms noted in Section 6.1.1, and the results will be exported, processed, and used to generate hazard and risk products. Model results required for FRA are outlined in the following section.

12.1 Simulations

For large HEC-RAS models with long runtimes, it is strongly recommended to use as many available performance cores as possible by running multiple plans in parallel. This can significantly reduce total simulation time when evaluating multiple recurrence intervals. HEC-RAS typically achieves optimal performance at approximately 8 performance cores per simulation. Increasing the number of cores beyond this threshold generally results in diminishing returns for a single run (Sabeti et al., 2024). As a result, machines with higher core counts should be used to execute multiple simulations concurrently across multiple instances rather than allocating all cores to a single plan.

12.1.1 Parallel Runs on a Single Machine

On a single machine, parallelization can be achieved by opening multiple instances of HEC-RAS and running different plans simultaneously. As a general guideline, the number of concurrent runs should be based on multiples of 8 performance cores. For example, a machine with 16 available performance cores can efficiently run 2 simulations in parallel Each HEC-RAS instance should open the same project and run a different plan. Care should be taken to ensure that plans are independent and that no two instances attempt to write the same output files.

12.1.2 Distributing Runs Across Multiple Machines

When the number of plans exceeds what can be efficiently run on a single machine, simulations should be distributed across multiple machines. To support this workflow, all plans must first be created and initialized within a single HEC-RAS project. Each plan should be run briefly (for example, approximately one minute) and then stopped. This step generates the required plan files and HDF structure so that all plans are fully recognized by the model. Once initialized, the entire project directory can be copied to multiple machines. Each machine is then assigned a subset of plans and runs them independently in parallel. No duplicate plans should be run across machines.

12.1.3 Combining Results Back into a Single Model

After all simulations are complete, the results must be consolidated into the original project. The HDF output files corresponding to each completed plan should be copied from the distributed machines back into the primary project directory, replacing the placeholder files generated during the initial short runs. Once all files are copied, the project should be opened, saved, and reloaded in HEC-RAS. At this point, all completed plan results will be synchronized within the project and will appear in RAS Mapper with the correct outputs and compute messages, as if they were run within a single environment.

12.2 DSS File and Reference Lines

The DSS output file included in the model folder should be submitted after it has been cleaned of all extra result layers that may have been generated during model development. Reference lines and reference points should be drawn at all calibration locations and locations of interest to extract flow and stage hydrographs into the DSS output file.

12.3 Shapefiles

Table 3 found in the Metadata and Data Source Catalog section of the report outlines all shapefiles that should be included in model submittals as well as accompanying data source information.

12.4 HEC-RAS Rendering Modes

For 2D RoM (pluvial) modeling, gridded raster datasets used for floodplain mapping and Flood Risk products should be exported using the sloping water surface rendering with Depth-Weighted Faces (Precip Mode) turned OFF. This approach produces a cleaner and more continuous pluvial floodplain, which is important for consistent mapping and for avoiding artificial fragmentation of flooded areas. When depth-weighting is enabled, very shallow rainfall runoff in flat overbank areas is mapped with strict rules about whether water can connect across small terrain variations. This often creates isolated wet patches that do not represent meaningful flooding but can inadvertently affect flood risk scores at receptors and complicate communication of results.

Depth-Weighted Faces were introduced to address a specific issue known as the cupping effect. In RoM models with steep transitions from shallow sheet flow into deeper channel flow, shallow water near cell edges can unrealistically pull the drawn water surface upward across a cell face toward a deeper neighboring cell. This can create artificial dips or distortions in the mapped water surface near channels. Depth-weighting reduces this problem by allowing the deeper flow to control how the water surface is drawn, which is most beneficial along channels and at sharp transitions from shallow to deep flow.

In this workflow, however, that benefit is not required, because fluvial and channelized areas are later superseded by a pseudo–steady-state riverine model. That riverine model defines the final channel water surfaces and floodplain edges, eliminating any channel-edge artifacts that might appear in the previous RoM results. With channel behavior resolved separately, keeping Depth-Weighted Faces turned on would only introduce unnecessary patchiness in flat overbank areas. Turning it off allows the sloping render to smoothly interpolate across minor terrain variations, resulting in clearer, more consistent pluvial floodplain maps that are better suited for district-wide flood risk assessment and communication.

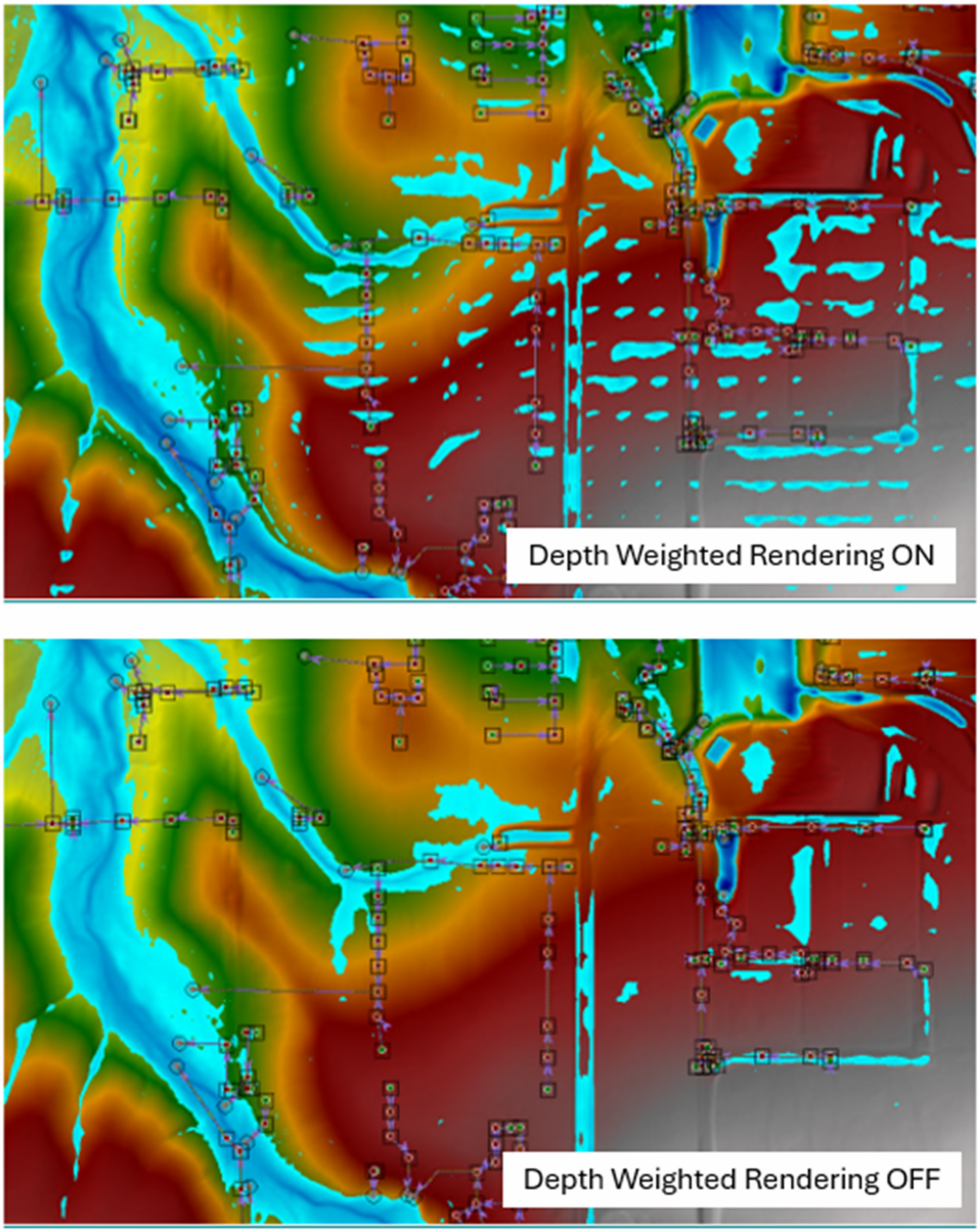
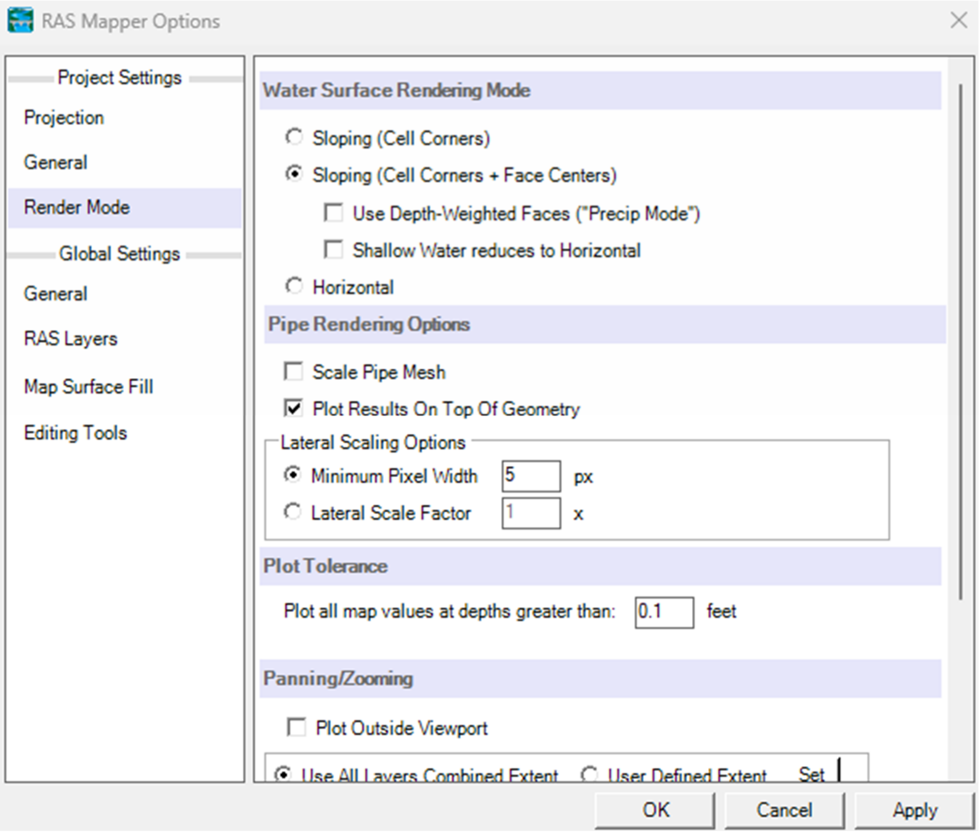


Figure 41. Comparison of Pluvial Flooding in Sloping Rendering Mode

12.5 Output Naming and File Structure

Maximum results (timestep agnostic) will be exported as rasters from RAS Mapper using the sloping method with depth-weighted faces turned off at a threshold of 0.1 feet (Figure 42), labeled in accordance with Table 21, and stored within the file structure shown in Figure 44. These rasters will be post-processed (Section 12.6) and then used to develop subsequent hazard products (Section 12.7) and flood risk scores (detailed under separate cover).



RAS Mapper Options

Project Settings
Projection
General
Render Mode
Global Settings
General
RAS Layers
Map Surface Fill
Editing Tools

Water Surface Rendering Mode
○ Sloping (Cell Corners)
● Sloping (Cell Corners + Face Centers)
☐ Use Depth-Weighted Faces ("Precip Mode")
☐ Shallow Water reduces to Horizontal
○ Horizontal

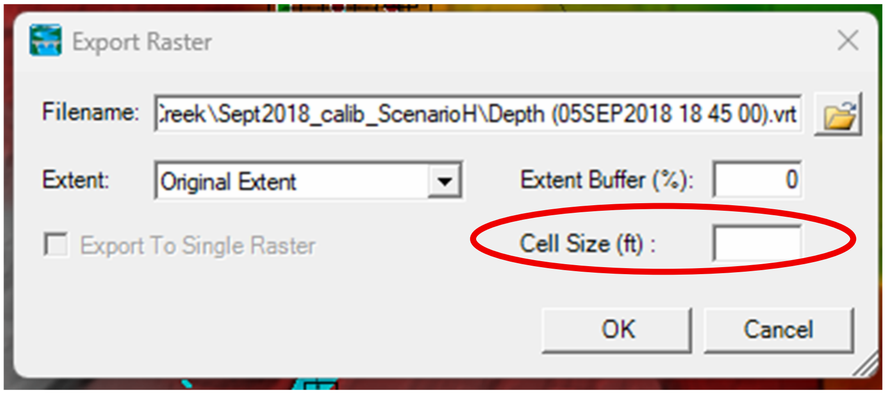
Pipe Rendering Options
☐ Scale Pipe Mesh
☑ Plot Results On Top Of Geometry
Lateral Scaling Options
● Minimum Pixel Width 5 px
○ Lateral Scale Factor 1 x

Plot Tolerance
Plot all map values at depths greater than: 0.1 feet

Panning/Zooming
☐ Plot Outside Viewport
● Use All Layers Combined Extent ○ User Defined Extent Set

Figure 42. Render Settings for Raster Exports

It is also important to note that there is an occasional error in exporting raster from HEC-RAS 7.0. If errors are produced when exporting the rasters or using the “Manage Result Map Layers” tool for a specific terrain it may be due to the automated cell size default not operating properly for that terrain layer. The current work around is to manually enter the cell size desired for the output raster as seen in the window in Figure 43



Export Raster

Filename: :reek\Sept2018_calib_ScenarioH\Depth (05SEP2018 18 45 00).vrt
Extent: Original Extent
Extent Buffer (%): 0
☐ Export To Single Raster
Cell Size (ft) :

Figure 43. HEC-RAS 7.0 Raster Export Input

Table 21. Model Output Rasters Naming (maximum)

Depth	Velocity	D×V
1yr_D	1yr_V	1yr_DV
2yr_D	2yr_V	2yr_DV

Depth	Velocity	D×V
5yr_D	5yr_V	5yr_DV
10yr_D	10yr_V	10yr_DV
25yr_D	25yr_V	25yr_DV
50yr_D	50yr_V	50yr_DV
100yr_D	100yr_V	100yr_DV
200yr_D	200yr_V	200yr_DV
300yr_D	300yr_V	300yr_DV
400yr_D	400yr_V	400yr_DV
500yr_D	500yr_V	500yr_DV
750yr_D	750yr_V	750yr_DV
1000yr_D	1000yr_V	1000yr_DV

Files shall be stored following the structure presented in Figure 44

```
Data_Bundle_Deliverable_Template/
├── hazard/                      # Postprocessed hazard outputs (rasters and vectors)
│   ├── aep/
│   │   └── aep_grid.tif
│   ├── depth/
│   │   └── *yr_D.tif
│   ├── dv/
│   │   └── *yr_DV.tif
│   ├── floodplains/
│   │   ├── 100yr_FP.gpkg
│   │   └── 500yr_FP.gpkg
│   └── velocity/
│       └── *yr_V.tif
│
├── model/                               # Hydraulic model inputs and supporting files
│   ├── streams.shp
│   ├── pipes_inventory.gpkg
│   ├── structure_inventory.gpkg
│   ├── hydroconnectors.gpkg
│   ├── model_boundary.gpkg
│   └── terrain.tif
├── risk/                   # Flood risk scores, threat grids, and risk driver maps
│   ├── FR_scores_ard/
│   │   ├── FR_structures.parquet
│   │   ├── FR_critical_facilities.parquet
│   │   ├── FR_roads.parquet
│   │   └── FR_trails.parquet
│   ├── FR_scores_presentation/
│   │   └── flood_risk.gdb/
│   │       ├── structure_risk
│   │       └── transit_risk
│   ├── threat_aep_grid_structures.tif
│   ├── threat_aep_grid_transit.tif
│   ├── risk_driver_structures.tif
│   └── risk_driver_transit.tif
```

Figure 44. Data Bundle Folder Structure

12.6 Post-Processing and Mapping Threshold

All exported rasters (Table 21) shall be post-processed prior to developing derivative hazard and risk products. These steps include:

1. Dry islands less than 2,500 sq. ft. should generally be filled. Disconnected inundation areas may be removed when supported by depth-based screening criteria, hydraulic connectivity, and engineering judgment demonstrating the area represents nuisance flooding rather than meaningful flood risk.
2. Measure depths within discontinuous flooding and delete areas whose maximum depth is less than 1 feet and whose average depth is less than 0.5 feet. Note: this draws upon logic for FEMA 1D floodplain mapping in what would constitute shallow flooding as Zone AO1.
3. Use streamlines associated with drainage areas greater than or equal to 130 acres (0.20 sq. mi.) to classify flooding associated with major drainageways as fluvial and remaining flooding as pluvial. Spatial intersection, Thiessen polygons, or similar GIS techniques may be used to define the classification boundaries. *Note: MHFD generally uses 130 acres as a 1D modeling threshold and denotes a typical denotation for major drainageways*
4. From MHFD Appendix B 2D Floodplain Mapping: Recommend a smoothing tolerance of 25 feet with the PAEK smoothing algorithm in GIS. This tolerance can be adjusted to make sure the resulting smooth boundary does not significantly alter the location of the flood boundary. PAEK smoothing should not exceed 50 feet. Simplifying/Generalizing vertices to 10 feet should be performed after smoothing is complete.

These four steps result in a more continuous layer of meaningful flood risk, which is an important process given the nature of RoM modeling which introduces water into literally every mesh cell. Furthermore, this process differentiates between pluvial and fluvial flooding, partially for local adoption purposes by local governments, but also as it relates to funding considerations for major drainageway improvements. The result of this process is illustrated by the three classifications in Figure 45, where blue denotes retained fluvial mapping; green denotes retained pluvial mapping; and black shows the underlying raw output that was removed via post-processing. These floodplain polygons are useful for visualization and processing of the risk assessment layers but are not meant for regulatory use. The polygon layers must be clearly labeled and identified with disclaimers to avoid misinterpretation.

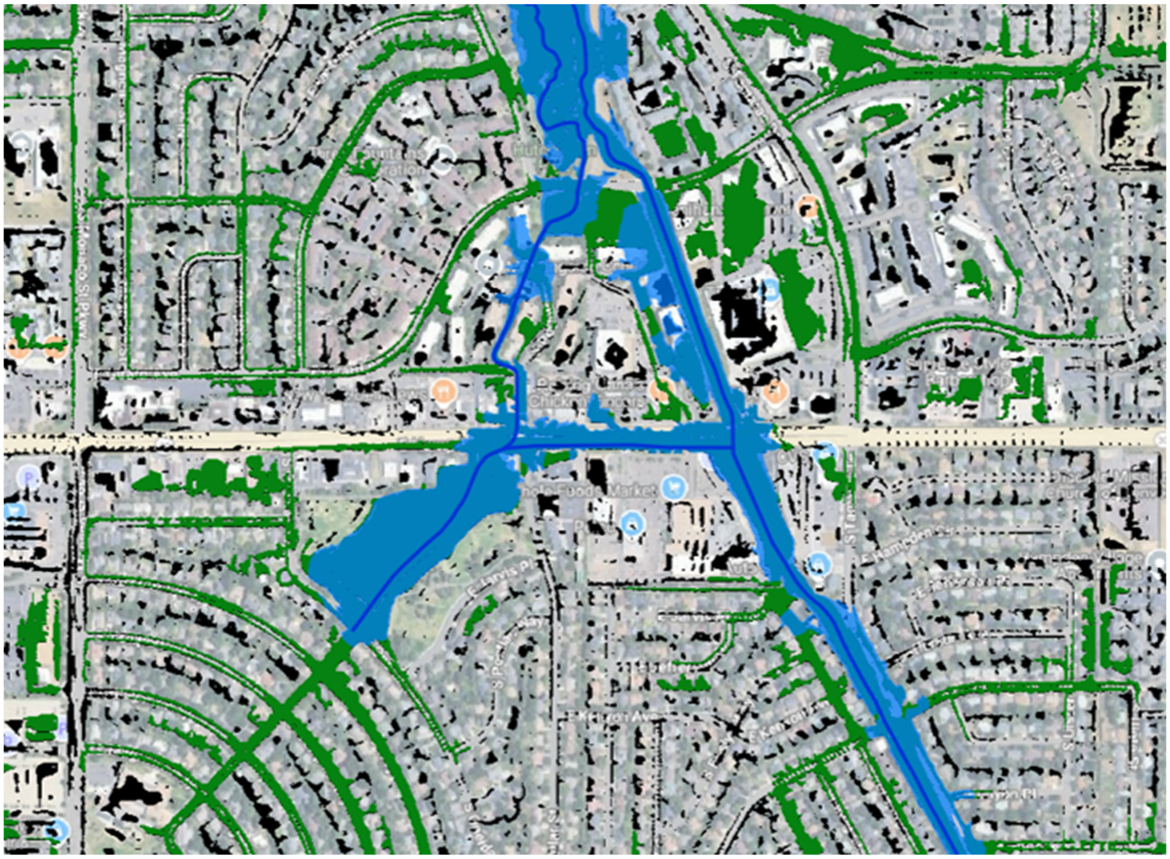


Figure 45. Post-Processed Floodplain Mapping Classifications

12.7 Hazard Products

This section is an abbreviated description of the end use for modeling products. The complete description of all hazard and risk products can be found in Section 4.2 of the Program Implementation Plan. Hazard products are then derived from the post-processed grids to include:

- Floodplains for the 1% and 0.2% events, converting rasters to polygons using standard GIS processes. Local governments may request that additional events be generated.
- A composite annual exceedance probability (AEP) map is generated from the grids through a summation of the bin weights (Table 22) when inundated (multiplied by 1 if wet and 0 if dry), as detailed in Section 4.1.3 of the Program Implementation Plan. This represents the frequency of flooding and has a maximum value of 1.0 if wetted during each event, such as a stream centerline.
- A similar approach is taken to develop a threat AEP map for both buildings and transit, except the bin weights (Table 22) are multiplied by a risk score between 0 and 3 based on the receptor building or transit criteria, which is detailed in Sections 4.1 and 4.2 of the Program Implementation Plan. The threat AEP maps how frequently and how dangerously it floods.

Table 22. Bin Weights by Recurrence Interval

Recurrence Interval	Probability	Bin Weight
1-year	1	0.25
2-year	0.5	0.40
5-year	0.2	0.20

Recurrence Interval	Probability	Bin Weight
10-year	0.1	0.080
25-year	0.04	0.040
50-year	0.02	0.015
100-year	0.01	0.0075
200-year	0.005	0.003333
300-year	0.00333	0.00125
400-year	0.0025	0.000667
500-year	0.002	0.000583
750-year	0.00133	0.00050
1000-year+	0.0010	0.001167

13 Quality Assurance

All models are required to undergo an internal QC process prior to submission to the MHFD or their designated contractor for external review. The internal review should follow the provided "MHFD Risk Framework QC Checklist" and the completed checklist must be included in model submittal. The intent of the QC review is to identify concerns and improve meaningful aspects of the RoM and PSS models in a manner that affect the end result. It is not intended to be a thorough extensive review for compliance purposes, but more of a "fatal flaw" review focused on hydraulic control. Once models are submitted to the District they will be reviewed again utilizing the same checklist to promote consistency for accuracy and completeness. If there is a future interest in a PSS model going regulatory, then it will need some modifications and is anticipated to undergo a comprehensive review for compliance. A comparison of the differences between a Flood Risk model and a regulatory model is included in Section 2.4 of the Program Implementation Plan.

A data collection inventory table following the format of Table 23 is required alongside model and report submission. The table should include all external data sources, format, dates collected, and notes that would be both helpful to a reviewer and for potential future studies that may leverage this information. The datasets listed in Table 23 should also be included in the final deliverable and verified for completeness.

Table 23. Example Data Collection Inventory for Completeness Check and Documentation

Item	Format	Source	Date Collected	Comment
Landcover	Raster	DRCOG/Planetary Computer	2020	1m resolution
Reservoir Rating Curves	PDF or FHAD Model	Local Governments (City of Arvada and Jeffco) 2003 As-Built Drawings, 2004 SWMM models, Denver water operations manuals.	2003, 2004	For all nine reservoirs
Storm sewer	Shapefile / Geodatabase	Local Governments (City of Arvada and Jeffco), MHFD SWMMS database	Obtained 3/16/2026	Contains nodes (inlets, outlets), pipes, and attributes
Soils	Shapefile	SSURGO	2026	
Structures	Hydraulic models and As-Builts	Effective FHAD models for Leyden and Ralston creek. Olsson draft FHAD models for Van Bibber Creek	Effective FHADs 2003, draft FHADs from Olsson 2026	
Base Terrain	Raster/DEM	USGS DRCOG 2022	2022	1m resolution
Supplemental Terrain	PDF for as-builts CSV or shapefile for survey	Local Government	2003	NAD83, NAVD88
Other Features (e.g., Levees)	PDF, Word, Excel	USACE, Local Government	2010	

14 Relevant FEMA Standards

The FEMA G&S relevant to the Risk Framework program is outlined below. In general, FEMA guidance on 2D modeling is based on previous 1D hydraulic modeling standards and the 2D standards are left with flexible interpretations to accommodate innovations that occur in the modeling space. Also, FEMA has intentionally avoided prescriptive guidance as it needs to accommodate a national scale with diverse physiogeographic settings, varied land use, and multiple modeling platforms. Therefore, most sections included in this guidance document do not have a corresponding section within the FEMA G&S as this guidance document aims to be more prescriptive due to the focused nature of the Flood Risk program.

The following section outlines the main topics FEMA guidance outlines for 2D modeling and the current standards and recommendations. This section is intended to be used as a correlative reference, but not as guidance. The guidance outlined specifically for MHFD Risk Framework watersheds should be followed as it typically follows FEMA standards but further expands upon them to encourage uniform submittals.

14.1 Terrain

Terrain standards outlined in the FEMA G&S must be followed for the Flood Risk framework analysis. The following items are the major takeaways from the guidance document, but it is recommended to review all terrain standards to ensure compliance.

- SID 44 – Bare Earth Terrain Processing:
    - Terrain datasets for flood modeling must be processed to bare earth in floodplain areas requiring hydraulic modeling.
- SID 43 – Vertical Accuracy Requirements:
    - Terrain data must meet FEMA’s documented vertical accuracy specifications based on flood risk and terrain slope. For high-risk, flat urban areas, this means a fundamental vertical accuracy of ≤24.5 cm and a nominal pulse spacing of ≤2 meters.
- SID 42 – Survey Certification:
    - All ground and structure surveys incorporated into terrain or bathymetry datasets must be certified by a registered professional engineer or licensed land surveyor.
      Note: this is not a requirement for MHFD Flood Risk structure connections.
- SID 416 – Consistency of Source Data:
    - Depth and analysis grids used in hydraulic modeling must share the same terrain and bathymetry source datasets as the engineering models.

14.2 Hydraulic Structures

There is limited data specific to 2D Hydraulic structures in the FEMA Risk Mapping, Assessment, and Planning G&S. The hydraulic structure requirements for the risk framework project are similar but provide additional flexibility so as not to require extensive data collection as these models are not inherently built to regulatory standards. Examples of deviations include the ability to use as-built information rather than surveyed; local government culvert databases where as-builts are not available;

and desktop assessment as a last resort. If the intention is to update the model to become regulatory these structures will have to be revisited and surveys updated to meet FEMA guidance.

Guidance for Flood Risk Analysis and Mapping Required Documentation

- The FEMA Mapping Partner must provide full documentation for every modeled structure, including:
    - The data sources for the structure's geometry and hydraulics.
    - The derivation method for the rating table.
    - All items required under Section 2.2.3 (Hydraulics: One-Dimensional Analysis – Hydraulic Structures).
- SID 42 Data Requirements
    - Structure modeling must be supported by surveyed channel geometry and structure measurements to ensure correct routing of flow through bridges, culverts, and other features.

14.3 Boundary Conditions and Initial Conditions

Boundary conditions and initial conditions are intentionally left slightly open ended in the FEMA G&S. The initial condition and boundary requirements outlined in the MHFD guidance provide additional detail and methodology and should be followed for the Risk Framework program.

- Establishment
    - Initial conditions in 2D hydraulic models must be established through preliminary warm-up simulations until stable, reasonable starting values are achieved.
- Required Evaluation
    - Mapping Partners must evaluate and document the simulation history and justify that the chosen initial conditions are reasonable before performing regulatory mapping runs.
- Antecedent Moisture and Recent Storm History
    - Rainfall-runoff modeling should incorporate realistic antecedent conditions (moisture state, reservoir levels, etc.) and simulate a typical recent storm history to set appropriate starting conditions.
    - Mapping Partners must analyze alternative realistic storm histories and document the sensitivity and implications for flood frequency assignment.
- Seasonal Conditions
    - In regions with distinct wet/dry seasons, Mapping Partners should use average wet-season conditions (average rainfall or average channel flow) when establishing initial WSEs used for mapping.

14.4 Calibration

FEMA guidance is vague on calibration metrics and slightly more specific on methodology. The risk framework guidance on calibration methods aligns closely with the methodology outlined in the FEMA G&S. The risk framework calibration standards, as of current guidance, are generally stricter and more specific than FEMA standards. Models should follow specific MHFD guidance on calibration methods and metrics outlined in Section 10 of this document.

- SID 59 – Calibration Using Observed Data:
  - H&H analyses must be calibrated using data from well-documented flood events, if available.
- SID 61 – Documentation of Calibration Process:
  - The calibration process, including input data, parameter adjustments, and justification for changes, must be thoroughly documented to ensure reproducibility.
- SID 62 – Agreement with Regulatory Products:
  - Calibration must ensure that modeled results align with observed data and regulatory products and are supported by sound engineering judgment.

15 References

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