



San Antonio River Basin Regional Modeling Standards for Hydrology &Hydraulic Modeling





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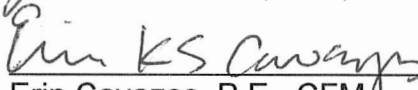
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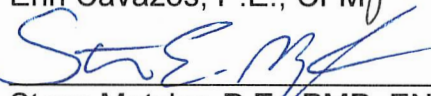
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1 INTRODUCTION

The San Antonio River Basin Regional Modeling Standards for Hydrologic and Hydraulic Modeling was developed in 2006 to support the regional flood studies prepared for the Federal Emergency Management Agency's (FEMA) Flood Map Modernization (Map Mod) program. These standards serve to provide consistent methodology for regional Hydrologic and Hydraulic (H&H) modeling within the San Antonio River Basin (SARB). They are intended to complement FEMA's Guidance and Standards for Flood Risk Projects. Relevant local, state, and federal modeling standards should always be followed for the design of flood mitigation projects.

The information included herein provides additional standards and guidelines specific to the San Antonio River Basin. These standards apply to all topographic, planimetric, survey, coefficient, and parameter data used to create or update regional hydrologic and hydraulic (H&H) models. Deviation from these standards may be accepted if accompanied by proper documentation and reasonable justification.

1.1 BACKGROUND

As part of FEMA's Map Mod program, the regulatory floodplains shown on the Flood Insurance Rate Maps (FIRMs) within the SARB were digitized and, in many cases, restudied between 2003 and 2010 producing the Digital Flood Insurance Rate Maps (DFIRMs) and DFIRM geospatial data for Bexar, Wilson, Karnes and Goliad Counties. Hydrologic models were developed for each watershed and hydraulic models were developed for major streams. This set of regional models is maintained and updated by the San Antonio River Authority (the River Authority) in coordination with the communities in the river basin and FEMA.

The H&H models, geospatial data, and documentation are available through the River Authority's website or by request. Documentation includes details about the development of the DFIRM H&H models and geospatial data and should be used as resources when working with the DFIRM H&H models and geospatial data.

1.2 MODEL UPDATES

The DFIRM H&H models and geospatial data on file with the River Authority shall be used as the basis for floodplain mapping and impact assessments within the SARB. All new models and modifications to existing models shall conform to the relevant standards and documentation requirements. Inclusion of future conditions may be added to existing models due to changes in political jurisdiction or Special Flood Hazard Area (SFHA) classification.

1.2.1 Hydrologic Models

When modifying a regional DFIRM hydrologic model, distinction shall be made between the models used to develop Flood Insurance Study (FIS) peak flows and design hydrology models used to capture details of a proposed project. Regional models should be developed based on hydrologic methods outlined in this SARB Regional Modeling Standards document. Design models would be required to meet local design guidelines that may specify different hydrologic methods. Engineers are required to evaluate influence of proposed projects on the regional models.

When beginning a new study, the engineer should check the River Authority's Digital Data & Modeling Repository (D2MR) for active Letter of Map Changes (LOMC) projects in the area. In addition, the engineer should document if LOMCs exist for their project area and whether those LOMCs were incorporated into the project analysis.

If a regional hydrologic model exists, design hydrologic models should be based on a truncated version of the regional model that can be modified and refined as necessary to represent pre- and post-project conditions. Truncation should occur at FIS nodes (the junction used to develop FIS peak flows). The

entire regional model subbasin(s) containing the main project area should be represented in the design hydrology model¹, but inflow hydrographs from the regional model may be used to represent upstream areas.

The difference between the pre- and post-project conditions peak flows at hydrologic nodes downstream of the project area should be used to measure the impacts of the project. If peak flows change by more than ± 5 percent, the project is considered to have an impact on hydrology. If the resulting hydrograph volume increases from pre- to post-project conditions at hydrologic nodes, coordination with the River Authority and local Floodplain Administrator (FPA) is required to determine if this change is considered an adverse impact. In addition, the allowable peak flow tolerance for a study may be reduced below 5 percent on a project-by-project basis at the discretion of the River Authority to mitigate regional hydrologic impacts.

Once a project's hydrologic impacts have been documented based on the design hydrologic model, engineers should modify the original regional model to reproduce the relative impact of the proposed project as described below. Changes should be made using the same software version as the original hydrologic model unless sufficient justification is provided.

Regional hydrologic models should be updated in the following cases:

1. If a proposed project incorporates a regional detention facility and/or diversion project, the regional detention facility and/or diversion project should be incorporated into the regional hydrology model.
2. If design model shows an impact on hydrology (producing a change in peak flows at any hydrologic node by more than ± 5 percent), the regional hydrology model subbasin and/or reach parameters should be modified to reflect the proposed project changes.
 - a. If the proposed project splits a drainage area or moves a drainage divide, the model subbasin area and associated subbasin parameters (e.g., loss parameters, land use, or impervious cover) may need to be updated to reflect that change. If the regional model uses Green and Ampt loss method, refer to Section 3.4.2.2 (Using the Green and Ampt Method) to verify or update calibration parameters.
 - b. If the proposed project physically changes surface conditions, the regional hydrology model loss method calculations and surface method calculations should be updated to reflect changes in land use and impervious cover.
 - c. If the proposed project changes the longest flowpath for the regional basin, the regional hydrology model transform method calculations should be updated.
 - d. If the proposed project changes the hydraulic parameters of a stream modeled as a reach in the regional hydrology model, the regional hydrology model reach routing method should be updated.
 - e. If applicable, refer to regional hydrology report for any alterations to subbasin parameters for calibration.

If the resulting peak flows vary more than ± 5 percent from the hydrologic node peak flows, the FIS should be updated to reflect the updated flows, water surface elevations, and floodplain extents. Regardless of whether the FIS is updated, the revised regional model should be submitted to the River Authority as the project conditions model for reference with subsequent studies in the area.

If no regional model exists, a design model following the methods outlined in this SARB Regional Modeling Standards document should be created based on the proposed project conditions and submitted to the River Authority for reference with subsequent studies.

¹ If the project area is located primarily in one subbasin with a small area crossing into an adjacent subbasin and if the area pertaining to the adjacent subbasin is less than 2% of the subbasin's total area, it is not necessary to include the entire adjacent subbasin in the design model.

1.2.2 Hydraulic Models

Regional DFIRM hydraulic models may not be truncated. They may be updated to a more recent analysis software version or converted to an alternative software package with sufficient justification and proper documentation. Any changes to the effective model structure or software version will require a comparison between the effective and duplicate effective models in the project study documentation. The use of an alternative modeling platform requires an explanation in the documentation on why the effective modeling platform was inappropriate for the model reach and why the proposed modeling platform should be used.

1.3 REGULATORY DESIGN EVENTS

The 1-percent annual chance (AC) existing conditions and the 1-percent AC future conditions storm events are the primary regulatory storm events for communities within the SARB. All models should include the 10-, 4-, 2-, 1-, and 0.2-percent AC existing conditions storm simulations. Studies within Bexar County shall include the 1-percent AC future conditions event in addition to those listed previously. Other communities may request the 1-percent AC future conditions event to be included in studies under their jurisdiction, if desired.

1.3.1 Existing Conditions

Existing conditions models are used to evaluate the impacts of the current development state in the watershed, establish a baseline for design of regional flood control facilities, and estimate flood risk.

In certain cases, the regulatory entity may require the use of more frequent design events for purposes other than FEMA FIRM development. Other purposes could include stream morphology, infrastructure, or regional storm-water facility design where smaller events may be critical to the performance of the design. Smaller events may include, but are not limited to, 20-, 25-, 50-, or 100-percent AC storm events.

1.3.2 Future Conditions

The 1-percent AC future conditions storm event is sometimes used to delineate the 1-percent AC future conditions floodplain mapping in lieu of the 0.2-percent AC floodplain on FEMA FIRM panels.

Future conditions should reflect the planned fully developed scenario of the communities for the watershed under study. The City of San Antonio (COSA) Unified Development Code (UDC) requires the inclusion of the 4-percent AC ultimate development (future conditions) event plus freeboard (if higher than the 1-percent AC event without freeboard) for the design of concrete or grass-lined channels. Higher frequency future conditions events may be used for design at the discretion of the regulatory entity.

1.4 STEPS FOR MODELING TO ACCOUNT FOR A PROJECT

When construction activities may impact a regulatory floodplain, a Condition Letter of Map Revision (CLOMR) may be required. Construction activities affecting a floodplain may include grading in the floodplain (cut/fill), creating a road over a stream (culverts and/or bridges), additional stormwater conveyance (new storm drain systems or channel improvements), or impounding flow by building a pond. Table 1.1 outlines various common project types and which modeling elements they typically impact. A CLOMR is required by FEMA when a project may result in mappable changes to water surface elevations (WSEs) for an established floodplain. Note that local floodplain administrators or state agencies may also require a CLOMR for compliance with local floodplain management regulations. For example, the City of San Antonio requires a CLOMR for all proposed physical changes to the floodplain that may result in changes to the floodplain extents. Changes in modeled WSEs and/or floodplain extents may be a result of changes to structures, roughness, or topography (cut and fill) within the floodplain limits. Changes in modeled WSEs and/or floodplain boundary may also be a result of changes outside of the floodplain limits altering the contributing flows, such as changes in drainage area boundaries, land use (e.g., from a project development), or overall flow patterns.

The 5-percent rule for changes in peak flow, mentioned in Section 1.2.1, should be considered for projects outside of a floodplain.

Table 1.1 – Overview of Hydrologic and Hydraulic Updates by Project Type

Project Element	Hydrology	Hydraulics
Lot grading/Fill	• Drainage areas • Soil parameters • Land use	• Cross section geometry • 2D surface • Manning's n • Flow data
Paving or buildings with traditional roofs	• Land use • Impervious cover	• Flow data
Permeable pavement*	• Land use • Impervious cover (reduced amount compared with traditional pavement)	• Flow data
Buildings with green roofs*	• Land use • Impervious cover (reduced amount compared with traditional roofs)	• Flow data
Detention ponds (On-line)	• Drainage areas • Storage-elevation-discharge relationships • Verify limit of routing function and adjust if detention replaces part or all of the routing reach.	• Flow data
Bioretention*	• Drainage areas • Storage-elevation-discharge relationships	• Flow data
Detention ponds (Off-line)	• Lateral diversion elevation-discharge relationship from unsteady hydraulic model • Pond storage-elevation-discharge relationship	• Lateral structure and storage in unsteady model • Flow data
Stock ponds	• No reservoir element required	• Inline structure on berm with an upstream and downstream bounding section • Expansion/Contraction coefficients
Dam	• Storage-elevation-discharge relationships • Verify limit of routing function and adjust if detention replaces part or all of the routing reach • Routing reach should stop near end of detention pool to avoid double counting storage volume	• Inline structure on berm with an upstream and downstream bounding section. • Expansion/Contraction coefficients • Upstream section WSE and downstream flow based on hydrologic model output.

Project Element	Hydrology	Hydraulics
Weir Structure	• Lateral diversion with stage-discharge relationship from unsteady hydraulic model	• Lateral structure in unsteady model • Flow data
Channelization	• Hydrologic routing function with output from hydraulic model (iterative)	• Cross section geometry • 2D surface • Manning's n • Hydrologic routing function
Vegetative swales* - if these are a modification to stream with a mapped floodplain.	• Hydrologic routing function with output from hydraulic model (iterative)	• Cross section geometry • 2D surface • Manning's n • Hydrologic routing function
Bridge	• Hydrologic routing function with output from hydraulic model (iterative)	• Structure data – deck, pier, and abutment geometry, low flow, and high flow computations • Cross-section data – ineffective flow areas, contraction and expansion losses, geometry if modified
Culvert	• Hydrologic routing function with output from hydraulic model (iterative)	• Structure data – type, shape and size of culvert, length of culvert, manning's roughness coefficient, entrance/exit loss coefficient, culvert invert elevations, roadway (deck) elevations • Cross-section data – ineffective flow areas, contraction and expansion losses, geometry if modified

*Refer to the Low Impact Development (LID) Design Manual for guidance on modeling LID for design models (the focus of the table above is on regional modeling).

1.4.1 Steps to Modify Hydrologic Models

1. **Effective Model:** Download the best available Effective model (D2MR or FEMA or best available model). If there is not a model available, a new existing conditions model should be created following the approach discussed in Section 1.4.2.
2. **Duplicate Effective Model:** Check if the modeling methodology and/or software used in the Effective model is appropriate.
 - a. If yes, run the model on your computer (*to create the Duplicate Effective model*)
 - b. If no, update software and document reasons why effective model software/approach was not appropriate.
3. **Corrected Effective Model:** Review the previously created Duplicate Effective model elements and parameters. Correct any errors in the duplicate effective model, add or adjust drainage area boundaries, or update topographic information or subbasin parameters (i.e., soil, land use, and transform parameters) to better reflect the physical conditions of the area at the date of the effective study. When considering these factors, incorporate any changes to reflect any missing

structural or site-specific changes which may have occurred *prior to* the date of the effective study. Any structural or site-specific changes occurring *after* the date of the effective study should be excluded. Consider whether additional changes are necessary to compare impacts with the revised or post-project model (such as subdividing a subbasin to model a proposed detention pond). After reviewing the Duplicate Effective model and parameters, are any changes necessary to better reflect the effective conditions at the date of the effective study?

- a. If yes, update the model (*to create the Corrected Effective model*) and note changes in the narrative.
 - b. If no, proceed with using the Duplicate Effective model as the Corrected Effective model. This can be documented in the report; a separate model run is not necessary.
4. **Existing/Pre-Project Model:** Review the previously created Corrected Effective model elements and parameters. Verify if there any structural or site-specific changes (such as a new TxDOT bridge) occurring after the date of the effective study that need to be accounted for in the model(s). After reviewing the Corrected Effective model and parameters, are any changes necessary to better reflect conditions after the date of the effective study?
- a. If yes, update the model (*to create the Existing/Pre-Project model*) and document changes in the report.
 - b. If no, proceed using the Corrected Effective model as the Existing Conditions/Pre-Project model. This can be documented in the report; a separate model run is not necessary.
5. **Proposed/Post-Project Model:** Create Proposed Project (CLOMR) or Post Project (LOMR) model. Table 1.1 can be used to understand what elements of the hydrologic model needs to be updated depending on what elements are included in the project.

1.4.2 Steps to Create New Hydrologic Models

The process for creating a new hydrologic model is outlined in Figure 1-1 and the list below.

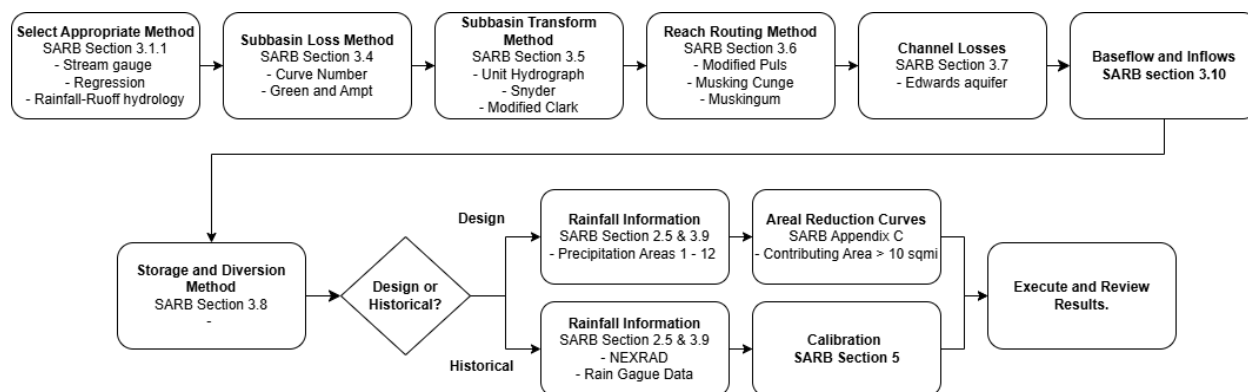


Figure 1-1 – Steps to Create New Hydrologic Model

1. Is Regression Method acceptable for your project area per Section 3.1.1.2 (Regression Equations)?
 - a. If yes, document reasons why Regression Method is acceptable and perform calculations. Do not need to follow subsequent bullets for Section 1.4.2 (Steps to Create New Hydrologic Models).
 - b. If no, develop Rainfall-Ruoff hydrology using latest version of HEC-HMS and follow subsequent bullets for Section 1.4.2 (Steps to Create New Hydrologic Models).
2. Are there hydrologic models for nearby watersheds that include subbasin elements?
 - a. If yes, determine subbasin loss methods (e.g., Curve Number or Green and Ampt) and transform methods (e.g., SCS or Snyder) are used and use same subbasin methods.
 - b. If no, select appropriate subbasin loss method following Section 3.4 (Loss Method), and select appropriate subbasin transform method following Section 3.5 (Hydrograph).

3. Are there hydrologic models for nearby watersheds that include reach elements?
 - a. If yes, determine reach methods are used (e.g., Lag, Muskingum-Cunge, or Modified Puls) and use same reach methods.
 - b. If no, select appropriate reach methods following Section 3.6 (Routing).
4. Delineate drainage areas based on latest detailed topographic data. Consider both the pre- and post-project needs. Note, if a project condition requires a divided subbasin, a pre-project conditions is needed with the same divided subbasin to ensure a 1:1 comparison for the project.
5. Calculated subbasin loss parameters for each drainage area delineated for existing and proposed conditions based on latest land cover information following Section 3.4 (Loss Method).
6. Calculate subbasin hydrograph (transform) parameters for each drainage area delineated for existing and proposed conditions following Section 3.5.1 (Method Selection).
7. Is attenuation between delineated subbasins expected to be significant?
 - a. If yes, identify limits of reach elements and calculate reach routing parameters for each reach element following Section 3.6 (Routing).
 - b. If no, reach elements may be left out of hydrologic model unless subbasins are in the Edwards Aquifer Recharge Zone.
8. If subbasins are located in the Edwards Aquifer Recharge Zone, document, and incorporate channel losses into reach elements in HEC-HMS following Section 3.7 (Channel Losses)
9. If detention ponds are within the delineated subbasins, refer to Section 3.8 (Storage and Diversions) for incorporating them into the HEC-HMS model.
10. Create Meteorological Models in HEC-HMS following Section 2.5 (Precipitation Data) and Section 3.9 (Rainfall).
11. If contributing drainage areas to the project location are greater than 10 square miles, Areal Reduction must be incorporated into hydrologic analysis. Refer to Appendix C for Areal Reduction Curves.
12. If baseflows are greater than 5 percent of the peak flow for the design storm event, baseflow should be included in the analysis and the source of baseflow data should be documented.
13. If historical or observed information is available, follow Section 5 (Calibration) for calibrating hydrology model.
14. Execute hydrological model for appropriate storm events and follow Section 3.11 (Documentation) for documenting the new hydrologic analysis.

1.4.3 Steps to Modify Hydraulic Models

1. **Effective Model:** Download the best available Effective model (D2MR or FEMA or best available model). If there is not a model available, a new existing conditions model should be created following the approach discussed in Section 1.4.4.
2. **Duplicate Effective Model:** Check if the modeling methodology and/or software used in the Effective model is appropriate.
 - a. If yes, run the model on your computer (*to create the Duplicate Effective model*)
 - b. If no, update software and document reasons why effective model software/approach was not appropriate.
3. **Corrected Effective Model:** Review the previously created Duplicate Effective model elements and parameters. Consider any errors in the duplicate effective model, add, or realign cross sections, or update topographic data or cross section data (i.e., roughness coefficients, expansion/contraction coefficients, and ineffective flow areas) to better reflect the physical conditions of the area at the date of the effective study. When considering these factors, incorporate any changes to reflect any missing structural or site-specific changes which may have occurred *prior* to the date of the effective study. Any structural or site-specific changes occurring *after* the date of the effective study should be excluded. Consider whether additional changes are necessary to compare impacts with the revised or post-project model (such as adding additional cross sections to model a proposed bridge). After reviewing the Duplicate Effective model and parameters, are any changes necessary to better reflect the effective conditions at the date of the effective study?

- a. If yes, update the model (*to create the Corrected Effective model*) and note changes in the narrative.
 - b. If no, proceed with using the Duplicate Effective model as the Corrected Effective model. This can be documented in the report; a separate model run is not necessary.
4. **Existing/Pre-Project Model:** Review the previously created *Corrected Effective model elements and parameters*. Verify if there any structural or project-specific changes (such as a new TxDOT bridge) occurring after the date of the effective study that need to be accounted for in the model(s). After reviewing the Corrected Effective model and parameters, are any changes necessary to better reflect conditions after the date of the effective study?
 - a. If yes, update the model (*to create the Existing/Pre-Project model*) and document changes in the report.
 - b. If no, proceed using the Corrected Effective model as the Existing Conditions/Pre-Project model. This can be documented in the report; a separate model run is not necessary.
5. **Proposed/Post-Project Model:** Create Proposed Project (CLOMR) or Post Project (LOMR) model. Table 1.1 can be used to understand what elements of the hydrologic and hydraulic models need to be updated depending on what elements are included in the project.

1.4.4 Steps to Create New Hydraulic Models

The process for creating a new hydraulic model is outlined in Figure 1-2 and the list below.

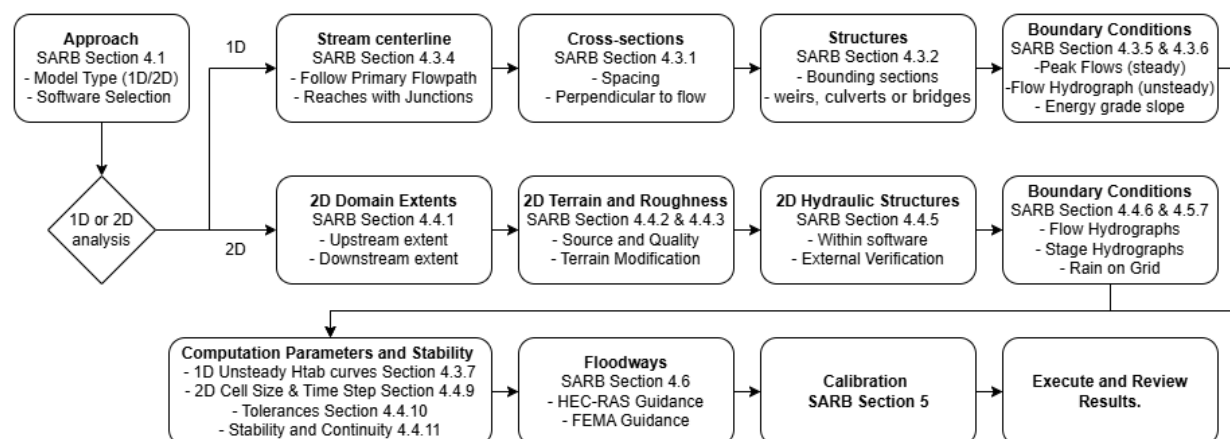


Figure 1-2 – Steps to Create New Hydraulic Model

1. Does the project require a 1D or 2D analysis to properly capture intent of the model? Refer to Section 4.1.
 - a. For a 1D analysis, refer to Section 4.3 Open Channel Riverine Models. Document why a 1D analysis is appropriate and if it is a steady or unsteady state analysis.
 - b. For a 2D analysis, refer to Section 4.4 Two-Dimensional Models. Document why a 2D analysis is appropriate.
 - c. Develop hydraulic model using latest version of HEC-RAS. Note, the use of other hydraulic modeling software is also acceptable depending on the modeling task, as long as the software is included on FEMA's *Hydraulic Models Meeting the Minimum Requirement of National Flood Insurance Program*.

2. For a 1D analysis:
 - a. Delineate stream centerline following the thalweg of the channel based on the latest detailed topographic data. Make sure stream centerline snaps to downstream reach. Refer to Section 4.3.4 (River Reaches and Junctions).
 - b. Layout hydraulic cross-sections following Section 4.3.1 (Cross-Sections). Note, if a project condition requires new cross-sections, a pre-project conditions is needed with the same cross-sections to ensure a 1:1 comparison for the project.
 - c. Layout hydraulic structures following Section 4.3.2 (Structures).
 - d. Add boundary conditions following Section 4.3.5 (Boundary Conditions) and Section 4.3.6 (Hydrologic Input).
 - e. For an unsteady 1D analysis refer to Section 4.3.7 (Additional Considerations for Unsteady 1D Riverine Models) for additional information on computation parameters and stability.
3. For a 2D analysis:
 - a. Establish a 2D domain following guidance in Section 4.4.1 (Domain Extents and Orientation).
 - b. Develop terrain and surface conditions following Section 4.4.2 (2D Terrain) and Section 4.4.3 (Roughness).
 - c. Layout hydraulic structures following Section 4.4.5 (2D Hydraulic Structures).
 - d. Add boundary conditions following Section 4.4.6 (Boundary Conditions) and Section 4.4.7 (Hydrologic Input).
 - e. For computation parameters and stability guidance refer to Section 4.4.9 (Cell Size and Time Step), Section 4.4.10 (Simulation Options and Tolerances), and Section 4.4.11 (Stability and Continuity Criteria).
4. If floodways are to be established for the project area refer to Section 4.6 (Floodways).
5. If historical or observed information is available, follow Section 5 (Calibration) for calibrating hydraulic model.
6. Execute hydraulic model for appropriate storm events and follow Section 4.7 (Documentation) for documenting the new hydraulic analysis.

2 SUPPORTING DATA

Quality supporting data for the development of numerical models is essential. Data typically used for model development includes, but is not limited to, topographic data for watershed delineation and cross-section geometry, survey data for hydraulic structures, and land use and soil data for the determination of loss rates.

2.1 TERRAIN DATA

Watershed modeling studies must use the best available topographic data for the area to be modeled and must meet the minimum standards outlined below. Data produced to a higher accuracy standard than minimum standards may be necessary or desirable for certain projects.

All topographic data used for hydraulic modeling and mapping must follow the FEMA Standards 40 – 49 relating to elevation data as well as the most current FEMA *Guidance for Flood Risk Analysis and Mapping: Elevation Guidance*. Vertical control should be established in project areas so that the digital surface model can be processed to meet the vertical standard.

A combination Light Detection and Ranging (LiDAR) data and survey was used to develop the DFIRM H&H models. When modifying existing models, topographic data for effective models may need to be supplemented or replaced with as-built drawings, if changes have occurred to the channel geometry since the effective date of the model. Additionally, new models and updates shall use a combination of the most recent LiDAR and survey data.

2.1.1 Aerial LiDAR

Aerial LiDAR data developed between 2013 and 2021 (or more recent) in cooperation with the Texas Water Development Board (TWDB), available for download through the Texas Geographic Information Office (TxGIO) Data Hub, will be considered the minimum acceptable standard for new models and model updates. The LiDAR data is accompanied by 3D breaklines of water features for use in developing surface triangulated irregular networks (TIN's). One-foot contours have been developed from LiDAR for some areas of the basin.

2.1.2 Drone LiDAR

On-site drone LiDAR may be used to provide greater accuracy and detail compared to other available terrain data. Drone-based LiDAR vertical accuracy should meet or exceed the vertical accuracy of the best publicly available LiDAR data for the study area available through the TXGIO DataHub or other data sources. Specific vertical accuracy standards for a study area are reported within the associated supplemental report and documentation included with each TXGIO LiDAR dataset and include Quality Level, point density, and root-mean-square deviation (RMSE) accuracy requirements. As an example, the 2022 supplemental report for the TXGIO “Bexar & Travis Counties Lidar” dataset states that the LiDAR project requirements meet or exceed the United States Geological Survey (USGS) Quality Level (QL) 2 specifications having a point density ≥ 4 pts/m² and an RMSE vertical accuracy ≤ 10 cm in Non-Vegetated Areas.

Where drone LiDAR is produced for a study, ground control and vertical accuracy checkpoints are required in accordance with FEMA *Guidance for Flood Risk Analysis and Mapping: Elevation Guidance*. Drone-based LiDAR data should also comply with the most recent USGS 3DEP LiDAR base specifications.

2.1.3 Terrain and Channel Surveys

Where new LiDAR is produced for a study, ground control and vertical accuracy checkpoints are required in accordance with FEMA *Guidance for Flood Risk Analysis and Mapping: Elevation Guidance*.

Where water obstructs LiDAR data collection and the area under water would represent active conveyance in the hydraulic model, surveyed channel cross-sections are required to establish the channel bathymetry for any of the following conditions:

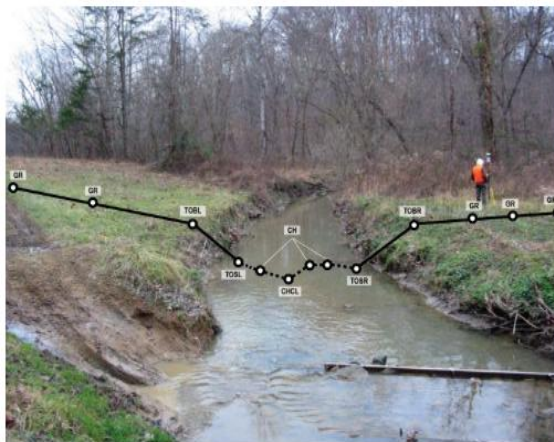
- Channel features are smaller than the LiDAR horizontal resolution
- There is heavy vegetation
- Channel walls are nearly vertical (cliffs)
- Additional detail is needed for streams where Base Flood Elevations (BFEs) are being revised or established.

If detailed topographic information is available and sufficient for hydraulic modeling and mapping outside of areas of standing water, the bathymetric survey data of channels should be used to modify the hydraulic modeling surface only below the water surface. The bathymetric data should then tie-in to the other detailed topographic data at the water surface edge.

In areas where the water obstructing the LiDAR represents normal standing water which would not represent active conveyance in the hydraulic model, bathymetry is not required. Where bathymetric data is included in the hydraulic modeling for areas of standing water, the area below the standing water should be modeled as an inactive area of non-conveyance in the hydraulic model.

In areas where detailed topographic data is not available, especially in flat areas, terrain surveys for hydrologic modeling may be required to define a surface for hydrologic and hydraulic modeling and mapping.

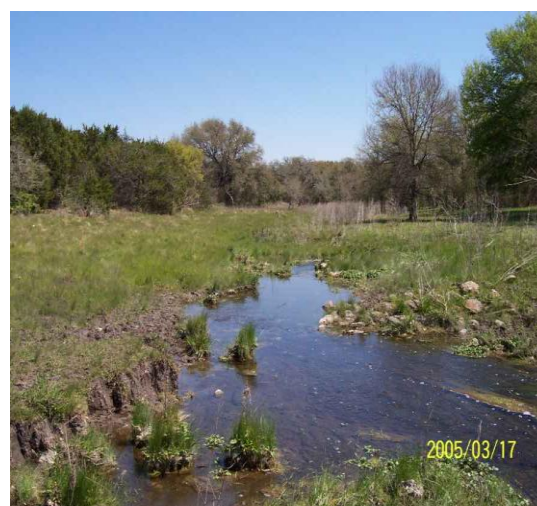
Channel cross-sections for hydraulic models should be surveyed at significant bends or locations where there are significant geometric changes so that the corresponding head losses will be properly incorporated. Photographs should be provided for surveyed cross-sections (Figure 2-1). These photographs should show the condition of the ground and vegetation (roughness) in the channel and overbanks in the vicinity of the cross-section and must be in accordance with FEMA *Guidance for Flood Risk Analysis and Mapping: Data Capture (Workflow Details)*. Refer to Section 2.4 for survey photography requirements and examples specific to structures such as bridges, culverts, dams, and levees.



Field Survey Locations (Image source: (FEMA, 2024, p. 23))



Upstream



Downstream

Figure 2-1 - Example Cross Section Photographs

Horizontal and vertical controls should be established using the Texas Department of Transportation (TXDOT) High Accuracy Reference Network (HARN) control monuments. If TXDOT's HARN system is not available, a different HARN system may be used but must be documented.

A standard system of survey codes helps to streamline the integration of survey data into the models and provides more consistency between survey submittals. Survey data should conform to the most current FEMA *Guidance for Flood Risk Analysis and Mapping: Data Capture and Data Capture (Workflow Details)*.

Channel survey data should be incorporated into hydraulic model cross sections where available. Any survey data used to create terrain surfaces or cross sections used in hydraulic models should be provided with the submittal of the hydraulic model. Information should be provided in the hydraulic model descriptions describing where survey data was used. For HEC-RAS models, a general note of the survey date and consultant performing the survey should be noted in the geometry file and plan file descriptions.

In addition, if specific cross sections are surveyed along the cross section cut line, the survey data source should be noted in the description for the surveyed cross sections specifically.

Any terrain surface files generated from surveyed data should be submitted with the hydraulic models. This may be submitted in the hydraulic model itself if the model allows the surface to be extracted. Otherwise, the terrain surface should be submitted in a separate file with the hydraulic model files. Acceptable geospatial file formats for submitting terrain data are described in Appendix E.

2.2 LAND USE

Land use data is a key component in the determination of loss rates for hydrologic modeling. The use of this data becomes more complicated due to changes in the state of development within urban areas and agricultural practices in rural areas with respect to time. Land use data is available for the entire SARB. However, the resolution and accuracy of this data can vary substantially.

When updated land use data is submitted in support of changes to a regional hydrologic model, it should meet the geospatial data standards presented in Appendix E. This allows the River Authority to maintain a land cover data set consistent with the data incorporated into the regional models.

2.2.1 Existing Conditions

The existing conditions land use information used in the SARB regional models included two primary components. The first was specific to Bexar County and the COSA. Within Bexar County, parcel and zoning data developed from tax information and aerial photography were used to define the existing conditions land use as of 2017. The percentages are intended to be representative of a particular land use type and will not precisely match the observed situation (i.e., aerial photography) in all cases. Likewise, the dataset is not intended to provide precise land use information detailed to the individual property level. The dataset will provide reasonable estimations for use in the derivation of hydrologic parameters. In some cases, the land classifications have been modified to more closely match the observed impervious cover rather than the true land use classification. These occurrences are fairly uncommon and primarily involve switches from commercial to industrial land use classifications.

The second component was USGS National Land Cover Data (NLCD). The NLCD was used to define existing conditions land use in the SARB outside of Bexar County. Figure 2-2 shows a map of the 2023 NLCD land use classifications across the entire San Antonio River Basin.

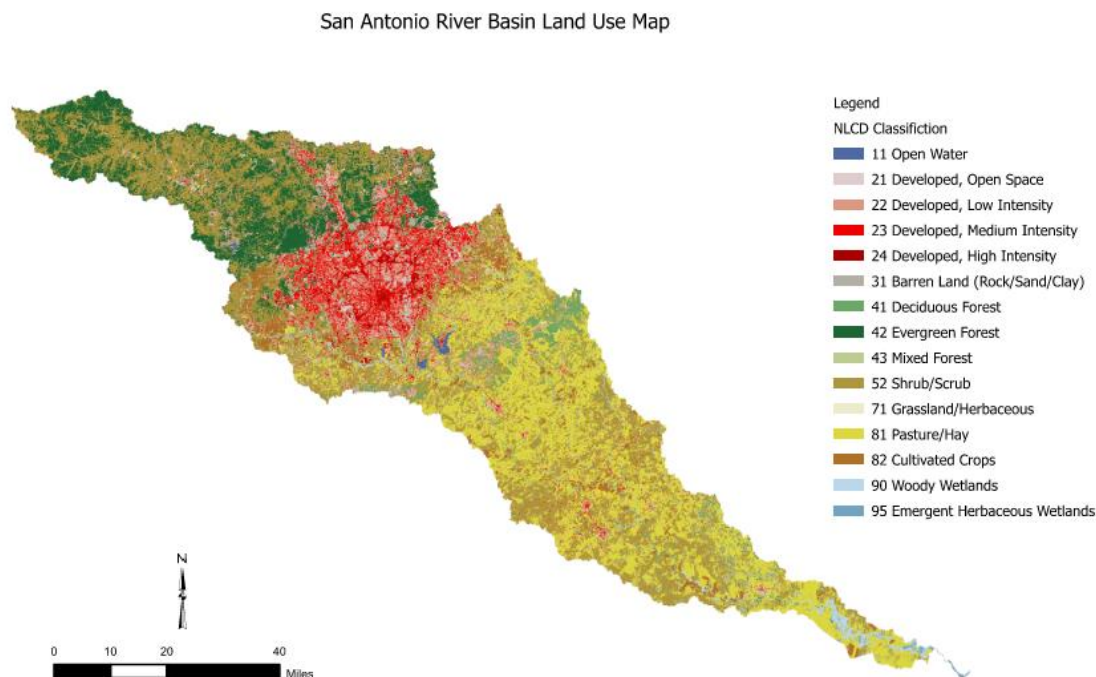


Figure 2-2 - San Antonio River Basin Land Use Map (2023)

Studies that develop new models or update existing models should use current parcel, zoning and aerial orthophotography data where available. Within Bexar County, parcel and zoning data is updated annually by the Bexar County Appraisal District (BCAD). Aerial orthophotography is updated biennially and maintained by the COSA. Outside of Bexar County, aerial orthophotography is available from the Texas Natural Resources Information System (TNRIS) at <http://www.tnris.org>.

If local land use information is not available, the most recent version of the USGS land cover data set may be used. A detailed description of this data is available on the USGS national mapping web site at <http://landcover.usgs.gov/usgslandcover.php>. Any detailed data developed in the future by other municipalities within the watershed should be used to supplement the USGS data. This is especially important in urbanized areas and regions of rapid development and land use change.

2.2.2 Future Conditions

Future conditions land use data is required to generate parameters for future conditions models. The River Authority commissioned the development of future conditions land use data in 2019. This dataset is used to represent future land use conditions throughout Bexar County in the scenario that development reaches its maximum potential or “full, ultimate build-out.” The land use conditions of the dataset are based on eighteen land use categories associated with varying degrees of impervious cover. Future land use conditions were estimated for the dataset using future planning information from the City of San Antonio (including planned future buildouts from San Antonio Tomorrow), TxDOT (including locations of major thoroughfares used to estimate potential locations for commercial areas) and other agencies (including state parks, preserves, conservancy districts, greenways, and other land areas). Additional information regarding the future land use dataset methodology can be found in the associated dataset documentation on the River Authority GIS Open Data website (SARA, 2019).

The geographic area represented by the updated “ultimate build-out” dataset also includes select watersheds located outside of Bexar County that will have an effect on the hydrologic environments within the River Authority’s jurisdiction. The dataset is not intended to provide a sufficient level (i.e., individual property level) of detail in either land use type or impervious coverage that would be used for determination of tax rates for storm water fees or any such similar uses.

Aside from the neighboring watersheds included in the updated “ultimate build-out” dataset, the creation of a future conditions land use dataset for areas outside of Bexar County was based on the USGS land use data modified based on population projections for the year 2060 developed for the TWDB Region L Water Plan.

Updates to the future conditions land use data should incorporate any additional data available from cities, counties, or other administrative entities within study area. If a study extends beyond the future conditions land use information available from local entities, the recommended approach for development of this data is to start with the USGS land cover as the base dataset and modify it using population projections developed for TWDB water use planning studies in the region assuming a 50-year planning horizon.

2.3 SOILS

Soils data is a key component in the development of loss rate parameters for hydrologic modeling. The Natural Resources Conservation Service (NRCS) is the primary supplier of this information. The NRCS converted their soil survey information into digital format. The digital data is produced in Soil Survey Geographic (SSURGO) format, designed for use in GIS software. Field mapping methods using national standards are used to construct the soil maps in the SSURGO database. Mapping scales generally range from 1:12,000 to 1:63,360. SSURGO is the most detailed level of soil mapping developed by the NRCS. The SSURGO data are organized by USGS 7.5-minute quadrangles within each county and are digital versions of the original soil survey maps. Additional information regarding SSURGO data can be found at <http://soils.usda.gov/>.

2.4 INFRASTRUCTURE DATA

When possible, all major structures in the stream reach to be modeled should be surveyed. The survey data shall conform to the most current FEMA *Guidance for Flood Risk Analysis and Mapping: Data Capture and Data Capture (Workflow Details)*, as well as the following:

- Flood Control Structures
 - Storage-elevation relationships must be defined with topographic data that have a minimum contour interval of 2 feet (1.2-foot vertical accuracy). Where an established volume and surface area relationship exists, it shall be used and verified against other available topographic information above the level pool elevation.
 - Studies involving the River Authority dams should obtain the current rating curve from the River Authority.
 - The outlet and spillway elevations should be surveyed or taken from as-built plans.
- Bridges and Culverts
 - Cross-sections should be surveyed at the upstream and downstream toe of the road embankment to define the bridge/culvert opening. When the channel geometry is consistent through the opening, one representative cross-section may be surveyed upstream and copied downstream. The surveyed cross-sections must identify all significant features of the bridge/culvert geometry (top of abutment, toe of slope, limits of riprap, etc.)
 - Flow lines (and centerlines if different) should be surveyed for each culvert barrel.
 - A cross-section should be surveyed along the centerline of the road over the bridge/culvert. If the centerline is not the highest point, the highest point must be identified and surveyed along with the centerline.
 - Any walls or bridge rails along the edges of the road should be surveyed (begin and end points and top elevations).
 - The low chord (top of bridge opening – often the bottom of the bridge beams and sometimes low-hanging utility lines) of the bridge shall be surveyed at both ends for flat bridge decks and at multiple points as needed to define sloping bridge decks or low chords with discontinuities (steps). The low chord shall be surveyed both on the upstream and the downstream side of the bridge.
 - The culvert headwall (back of curb edge downstream – where slope meets top of culvert or headwall above culvert centerline on downstream end for determining outlet projection, and culvert upstream crown – the highest point of the upstream end of a culvert) shall be surveyed along the culvert centerline plus the culvert wall with the edge of the culvert on both the upstream and downstream sides if the structure.
 - The wingwalls (wing wall – top face of each end of a wing wall or headwall on a structure to define height and length, and wing wall bottom – base of each end of a wing wall or head wall on a structure to define height and length) of the culvert shall be surveyed at both ends and at multiple

points as needed to define the top and bottom of the wall plus the angle of the wingwall. The wingwalls shall be surveyed on both the upstream and downstream sides of the structure.

- All bridge piers (pier – the up and downstream centerline of a pier) shall be surveyed at the centerline of the pier on both the upstream and downstream sides, along with the top and bottom elevations.
- Dams/Weirs
 - Cross-sections should be surveyed immediately upstream and downstream of the dam or weir.
 - A cross-section should be surveyed along the top of the dam or weir, capturing the highest elevations that form the dam/weir, and the shape of the weir.
 - The flow line elevation at the upstream and downstream end of any low-flow outfall should be surveyed.
 - Sufficient surveying should be performed to fully define any weir opening in the structure.
 - Refer to Section 4.3.2.4 (Dams and Stock Ponds) for hydraulic model geometry considerations when modeling dam/weir structures.
 - Refer to Section 4.3.5 (Boundary Conditions) for incorporating boundary conditions from the Hydrologic model into the Hydraulic model.
- Field Notes/Sketches
 - Adequate field notes must be provided to describe the size and shape of any surveyed opening, weir or spillway. A sketch of the surveyed structure should also be provided.
 - Any siltation or blockage of culvert or bridge openings should be noted and shown in a sketch and photograph.
 - The width of the roadway and any shoulder (paved or grass lined) should be indicated in the notes.
- Survey Photography
 - Photographs must be provided for any surveyed structure including, but not limited to, bridges, culverts, dams, and levees. Survey photographs should follow the guidelines outlined in the FEMA *Guidance for Flood Risk Analysis and Mapping: Data Capture and Data Capture (Workflow Details)*.
 - At a minimum, five photographs should be taken for each structure showing (1) the downstream channel (from the structure facing downstream), (2) the downstream face of the structure, (3) the overtopping cross section (oriented showing the direction of flow from left to right), (4) the upstream face of the structure, and (4) the upstream channel (from the structure facing upstream) (Figure 2-3).

- Photographs should include a description and either a date stamp or the date included with the description.
- Standardized field codes and comments as outlined in the most current FEMA *Guidance for Flood Risk Analysis and Mapping: Data Capture* and *Data Capture (Workflow Details)* should be used for all surveys associated with modeling projects.
- If sufficiently accurate as-built plans are available for a structure, they may be used in place of field survey. The use of any as-built drawings must be approved by the reviewing entity on a case-by-case basis before they are used to develop the model. The datum used to establish the elevations for the plans should be noted and elevations must be adjusted to match the modeling standards described in this document.

Figure 2-3 - Example Structure Photographs



Downstream Channel



Downstream Structure Face



Overlapping Cross Section



Upstream Structure Face



Upstream Channel

2.5 PRECIPITATION DATA

Precipitation values will be based on *NOAA Atlas 14 Precipitation-Frequency Atlas of the United States Volume 11: Texas* (Perica, et al., 2018). This publication supersedes the traditionally-used TP-40 (Hershfield, 1961) and Hydro-35 (Frederick, Myers, & Auciello, 1977) publications. Precipitation values have been summarized into twelve Precipitation Areas based on hydrologic boundaries as shown in Figure 2-4 – NOAA Atlas 14 Precipitation Areas. The values for the Precipitation Area are included in Table 2.1 through Table 2.12.

For historical events, Next-Generation Radar (NEXRAD) data is available from the National Oceanic and Atmospheric Administration (NOAA), National Climatic Data Center. Rain gauge data from the Edwards Aquifer Authority, Bexar County, the City of San Antonio, and the San Antonio River Authority is available upon request.

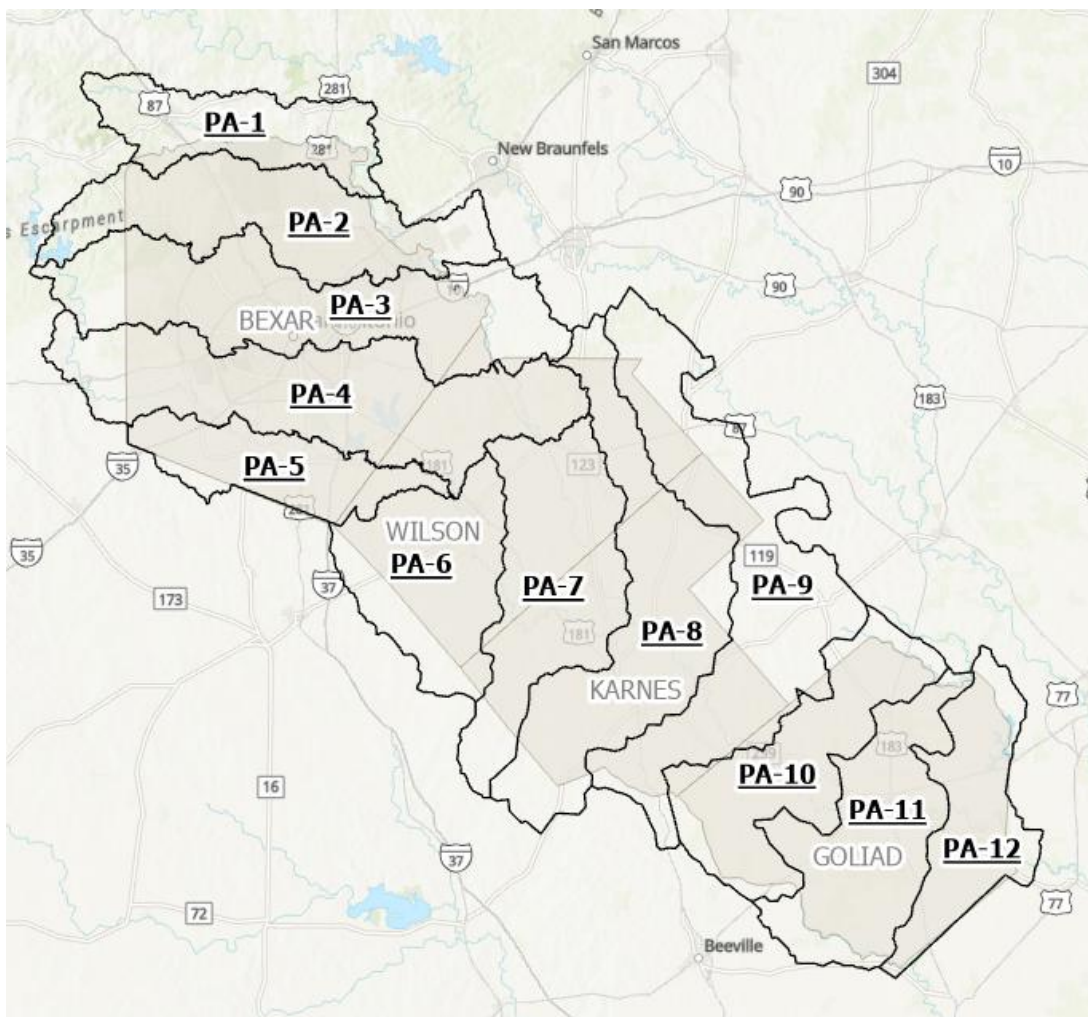


Figure 2-4 – NOAA Atlas 14 Precipitation Areas

Table 2.1 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 1

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.45	0.53	0.66	0.78	0.94	1.06	1.19	1.52
10 minute	0.71	0.84	1.06	1.24	1.50	1.70	1.90	2.40
15 minute	0.90	1.06	1.33	1.55	1.86	2.11	2.36	3.00
30 minute	1.27	1.50	1.87	2.17	2.60	2.93	3.27	4.19
1 hour	1.65	1.96	2.46	2.88	3.47	3.93	4.41	5.76
2 hour	1.98	2.42	3.09	3.69	4.57	5.28	6.07	8.26
3 hour	2.15	2.69	3.48	4.21	5.30	6.21	7.24	10.10
6 hour	2.46	3.16	4.15	5.09	6.54	7.80	9.23	13.26
12 hour	2.77	3.62	4.80	5.95	7.71	9.25	11.03	16.23
24 hour	3.12	4.12	5.51	6.86	8.93	10.75	12.87	19.14
2-day	3.57	4.72	6.35	7.92	10.34	12.44	14.88	21.98
3-day	3.89	5.14	6.91	8.60	11.20	13.46	16.06	23.47
4-day	4.15	5.45	7.31	9.09	11.78	14.11	16.77	24.33
7-day	4.75	6.15	8.18	10.08	12.93	15.34	18.07	25.80

Table 2.2 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 2

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.45	0.53	0.66	0.77	0.93	1.05	1.17	1.47
10 minute	0.71	0.84	1.06	1.24	1.48	1.68	1.87	2.33
15 minute	0.90	1.06	1.33	1.55	1.85	2.08	2.32	2.91
30 minute	1.28	1.50	1.86	2.16	2.58	2.89	3.21	4.07
1 hour	1.65	1.96	2.45	2.86	3.44	3.87	4.32	5.59
2 hour	1.97	2.41	3.08	3.67	4.52	5.20	5.95	8.03
3 hour	2.14	2.67	3.46	4.18	5.24	6.12	7.10	9.84
6 hour	2.44	3.13	4.11	5.05	6.45	7.66	9.04	12.90
12 hour	2.76	3.58	4.75	5.87	7.58	9.06	10.76	15.73
24 hour	3.10	4.07	5.42	6.73	8.74	10.47	12.49	18.44
2-day	3.53	4.64	6.21	7.72	10.02	12.03	14.34	21.00
3-day	3.84	5.03	6.74	8.36	10.83	12.95	15.38	22.32
4-day	4.10	5.34	7.14	8.83	11.38	13.57	16.05	23.10
7-day	4.69	6.03	8.00	9.81	12.50	14.76	17.29	24.44

Table 2.3 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 3

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.45	0.53	0.66	0.77	0.92	1.03	1.15	1.43
10 minute	0.71	0.84	1.05	1.23	1.47	1.66	1.84	2.27
15 minute	0.90	1.06	1.32	1.53	1.83	2.05	2.28	2.83
30 minute	1.28	1.49	1.85	2.14	2.55	2.85	3.16	3.96
1 hour	1.64	1.94	2.43	2.83	3.39	3.81	4.24	5.42
2 hour	1.96	2.38	3.04	3.62	4.44	5.10	5.81	7.75
3 hour	2.12	2.64	3.43	4.11	5.14	5.98	6.91	9.46
6 hour	2.42	3.08	4.05	4.95	6.31	7.45	8.74	12.36
12 hour	2.73	3.52	4.66	5.73	7.36	8.76	10.36	14.99
24 hour	3.07	3.99	5.30	6.55	8.45	10.08	11.97	17.52
2-day	3.49	4.54	6.05	7.48	9.66	11.54	13.69	19.90
3-day	3.79	4.92	6.56	8.10	10.43	12.41	14.68	21.14
4-day	4.04	5.22	6.95	8.55	10.96	13.01	15.32	21.86
7-day	4.63	5.90	7.79	9.51	12.05	14.16	16.51	23.11

Table 2.4 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 4

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.45	0.53	0.65	0.76	0.91	1.02	1.14	1.40
10 minute	0.71	0.84	1.04	1.22	1.46	1.64	1.82	2.22
15 minute	0.91	1.06	1.31	1.52	1.81	2.03	2.26	2.77
30 minute	1.28	1.49	1.84	2.12	2.52	2.82	3.12	3.87
1 hour	1.64	1.93	2.41	2.80	3.35	3.76	4.19	5.28
2 hour	1.95	2.37	3.02	3.58	4.38	5.02	5.70	7.51
3 hour	2.12	2.62	3.38	4.07	5.06	5.86	6.74	9.14
6 hour	2.41	3.05	4.01	4.88	6.18	7.27	8.48	11.87
12 hour	2.70	3.47	4.57	5.61	7.17	8.49	9.99	14.32
24 hour	3.03	3.91	5.18	6.38	8.19	9.73	11.50	16.69
2-day	3.44	4.45	5.91	7.28	9.36	11.14	13.17	18.98
3-day	3.73	4.82	6.40	7.88	10.10	11.99	14.13	20.18
4-day	3.98	5.12	6.78	8.32	10.62	12.56	14.74	20.87
7-day	4.58	5.80	7.63	9.27	11.69	13.68	15.89	22.05

Table 2.5 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 5

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.45	0.53	0.65	0.76	0.91	1.02	1.14	1.39
10 minute	0.71	0.84	1.04	1.22	1.45	1.64	1.82	2.19
15 minute	0.91	1.06	1.30	1.51	1.80	2.03	2.26	2.74
30 minute	1.28	1.49	1.83	2.11	2.51	2.82	3.12	3.83
1 hour	1.64	1.93	2.40	2.80	3.34	3.76	4.18	5.22
2 hour	1.95	2.36	3.01	3.57	4.36	4.99	5.67	7.41
3 hour	2.12	2.61	3.37	4.05	5.03	5.82	6.69	8.99
6 hour	2.41	3.04	3.98	4.85	6.12	7.18	8.36	11.60
12 hour	2.70	3.44	4.53	5.54	7.05	8.32	9.76	13.83
24 hour	3.02	3.87	5.11	6.27	8.01	9.48	11.15	15.95
2-day	3.43	4.40	5.83	7.15	9.14	10.82	12.72	18.05
3-day	3.73	4.78	6.32	7.74	9.86	11.64	13.63	19.16
4-day	3.98	5.07	6.69	8.17	10.36	12.18	14.21	19.81
7-day	4.58	5.74	7.52	9.10	11.38	13.24	15.27	20.92

Table 2.6 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 6

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.45	0.53	0.66	0.76	0.91	1.02	1.13	1.37
10 minute	0.72	0.85	1.05	1.22	1.45	1.64	1.81	2.17
15 minute	0.92	1.07	1.30	1.50	1.78	2.02	2.24	2.71
30 minute	1.29	1.50	1.82	2.10	2.49	2.80	3.10	3.76
1 hour	1.66	1.94	2.40	2.79	3.31	3.73	4.14	5.11
2 hour	1.97	2.37	3.01	3.55	4.32	4.93	5.56	7.17
3 hour	2.14	2.62	3.38	4.04	4.98	5.73	6.54	8.67
6 hour	2.41	3.04	3.98	4.83	6.06	7.07	8.18	11.24
12 hour	2.70	3.42	4.51	5.50	7.00	8.25	9.67	13.68
24 hour	3.01	3.81	5.05	6.19	7.98	9.43	11.17	16.13
2-day	3.43	4.25	5.71	6.96	9.16	10.77	12.89	18.59
3-day	3.71	4.62	6.14	7.55	9.73	11.74	13.51	19.67
4-day	3.96	4.92	6.52	8.00	10.27	12.45	14.01	20.55
7-day	4.63	5.64	7.40	9.00	11.36	13.32	15.50	21.55

Table 2.7 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 7

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.45	0.53	0.65	0.75	0.90	1.01	1.13	1.36
10 minute	0.72	0.84	1.04	1.20	1.44	1.63	1.81	2.16
15 minute	0.92	1.06	1.29	1.49	1.78	2.01	2.23	2.69
30 minute	1.29	1.49	1.81	2.09	2.47	2.78	3.09	3.76
1 hour	1.66	1.94	2.38	2.76	3.29	3.71	4.13	5.11
2 hour	1.98	2.37	3.00	3.53	4.29	4.90	5.54	7.13
3 hour	2.14	2.62	3.37	4.02	4.95	5.70	6.51	8.62
6 hour	2.42	3.04	3.98	4.83	6.06	7.07	8.18	11.25
12 hour	2.69	3.43	4.52	5.53	7.05	8.32	9.77	13.93
24 hour	2.95	3.84	5.09	6.27	8.05	9.60	11.43	16.66
2-day	3.22	4.30	5.80	7.14	9.19	11.06	13.37	19.44
3-day	3.54	4.66	6.22	7.71	10.03	12.20	14.15	20.74
4-day	3.77	4.97	6.62	8.17	10.63	12.99	14.77	21.77
7-day	4.43	5.65	7.45	9.10	11.57	13.66	16.00	22.46

Table 2.8 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 8

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.46	0.53	0.65	0.76	0.90	1.02	1.13	1.37
10 minute	0.73	0.85	1.04	1.21	1.44	1.63	1.81	2.17
15 minute	0.92	1.07	1.31	1.51	1.79	2.02	2.24	2.71
30 minute	1.30	1.50	1.83	2.11	2.49	2.80	3.10	3.77
1 hour	1.68	1.96	2.40	2.78	3.31	3.73	4.15	5.13
2 hour	2.00	2.39	3.09	3.55	4.31	4.92	5.56	7.18
3 hour	2.17	2.65	3.51	4.03	4.97	5.72	6.53	8.67
6 hour	2.45	3.07	4.01	4.85	6.08	7.10	8.23	11.34
12 hour	2.72	3.48	4.57	5.58	7.11	8.40	9.87	14.10
24 hour	3.00	3.92	5.16	6.37	8.22	9.78	11.60	16.93
2-day	3.32	4.46	5.91	7.32	9.56	11.40	13.60	19.83
3-day	3.60	4.71	6.31	7.86	10.31	12.06	14.62	20.91
4-day	3.83	4.99	6.68	8.34	10.95	12.53	15.32	21.64
7-day	4.46	5.71	7.55	9.26	11.79	13.95	16.36	23.03

Table 2.9 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 9

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.46	0.54	0.66	0.76	0.90	1.02	1.13	1.37
10 minute	0.73	0.85	1.05	1.22	1.45	1.63	1.81	2.17
15 minute	0.93	1.08	1.32	1.52	1.80	2.02	2.24	2.72
30 minute	1.31	1.52	1.85	2.13	2.51	2.81	3.11	3.79
1 hour	1.70	1.98	2.43	2.81	3.33	3.74	4.16	5.16
2 hour	2.02	2.42	3.04	3.57	4.33	4.93	5.57	7.23
3 hour	2.20	2.67	3.40	4.05	4.98	5.73	6.54	8.73
6 hour	2.49	3.11	4.03	4.87	6.11	7.13	8.27	11.45
12 hour	2.78	3.55	4.63	5.67	7.21	8.52	10.02	14.33
24 hour	3.10	4.00	5.30	6.51	8.39	10.01	11.89	17.35
2-day	3.47	4.57	6.10	7.52	9.80	11.79	14.08	20.53
3-day	3.73	4.89	6.52	8.12	10.74	12.55	15.20	21.62
4-day	3.99	5.19	6.92	8.60	11.47	13.10	15.99	22.45
7-day	4.62	5.90	7.80	9.55	12.17	14.40	16.89	23.72

Table 2.10 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 10

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.47	0.55	0.67	0.77	0.92	1.03	1.14	1.39
10 minute	0.75	0.87	1.07	1.24	1.47	1.65	1.82	2.19
15 minute	0.95	1.10	1.35	1.57	1.83	2.05	2.27	2.75
30 minute	1.35	1.56	1.90	2.18	2.56	2.86	3.16	3.84
1 hour	1.76	2.04	2.50	2.88	3.41	3.82	4.23	5.25
2 hour	2.11	2.51	3.13	3.66	4.43	5.03	5.67	7.33
3 hour	2.30	2.78	3.51	4.16	5.09	5.85	6.67	8.84
6 hour	2.62	3.24	4.17	5.01	6.26	7.31	8.46	11.60
12 hour	2.90	3.68	4.81	5.86	7.46	8.81	10.34	14.58
24 hour	3.21	4.14	5.49	6.79	8.72	10.44	12.36	17.71
2-day	3.56	4.62	6.25	7.86	10.13	12.41	14.77	20.93
3-day	3.79	5.03	6.78	8.46	11.26	13.24	15.98	22.33
4-day	4.04	5.34	7.19	8.95	12.03	13.85	16.83	23.25
7-day	4.71	6.07	8.07	9.89	12.59	14.85	17.38	23.95

Table 2.11 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 11

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.48	0.55	0.68	0.78	0.92	1.03	1.14	1.38
10 minute	0.76	0.88	1.08	1.24	1.47	1.65	1.82	2.19
15 minute	0.96	1.11	1.36	1.56	1.84	2.06	2.27	2.74
30 minute	1.37	1.58	1.92	2.20	2.59	2.86	3.17	3.86
1 hour	1.79	2.07	2.54	2.93	3.46	3.82	4.27	5.28
2 hour	2.16	2.56	3.19	3.73	4.50	5.10	5.74	7.38
3 hour	2.37	2.85	3.58	4.23	5.17	5.94	6.77	8.90
6 hour	2.71	3.34	4.28	5.13	6.40	7.47	8.64	11.72
12 hour	3.02	3.82	4.97	6.07	7.71	9.10	10.65	14.90
24 hour	3.34	4.32	5.73	7.08	9.10	10.91	12.90	18.24
2-day	3.74	4.92	6.55	8.24	10.70	13.05	15.55	21.67
3-day	3.96	5.29	7.15	9.00	11.42	13.99	16.76	23.11
4-day	4.21	5.63	7.57	9.56	11.87	14.69	17.69	24.11
7-day	4.93	6.35	8.43	10.33	13.17	15.56	18.18	25.09

Table 2.12 - NOAA Atlas 14 DDF Rainfall Values for Precipitation Area 12

Total Rainfall for Frequency (Inches)

Duration	100% AC	50% AC	20% AC	10% AC	4% AC	2% AC	1% AC	0.2% AC
5 minute	0.49	0.56	0.68	0.79	0.93	1.04	1.14	1.38
10 minute	0.77	0.89	1.09	1.25	1.48	1.66	1.83	2.18
15 minute	0.98	1.13	1.37	1.57	1.85	2.07	2.28	2.73
30 minute	1.40	1.61	1.94	2.22	2.60	2.90	3.19	3.85
1 hour	1.84	2.12	2.58	2.96	3.48	3.89	4.31	5.28
2 hour	2.23	2.63	3.25	3.79	4.55	5.17	5.81	7.37
3 hour	2.45	2.93	3.67	4.32	5.26	6.03	6.85	8.89
6 hour	2.81	3.45	4.40	5.26	6.54	7.61	8.77	11.73
12 hour	3.13	3.95	5.16	6.26	7.92	9.31	10.87	14.99
24 hour	3.44	4.50	5.96	7.35	9.44	11.22	13.16	18.45
2-day	3.71	5.15	6.90	8.68	11.29	13.53	15.83	22.00
3-day	4.09	5.48	7.50	9.49	12.00	14.54	17.26	23.57
4-day	4.36	5.83	7.95	10.13	12.56	15.31	18.25	24.66
7-day	5.21	6.66	8.85	10.84	13.73	16.11	18.73	25.71

2.6 STREAM FLOW AND STAGE DATA

Stream flow and stage data is available from the USGS National Water Information System for flood frequency analysis and/or model calibration. Historical data may also be available for the gauge stations which are no longer active. <https://waterdata.usgs.gov/nwis>

Stage data may also be available from the City of San Antonio and Bexar County high water detection system is available upon request.

3 HYDROLOGIC MODELING

3.1 APPROACH

3.1.1 Permitted Methods

FEMA provides guidance related to three hydrologic analysis methods for determining peak flows: stream gauge analysis, regression equations, and rainfall-runoff models. Due to the limited availability of gauge data in the SARB, rainfall-runoff models are the most commonly used method. Other methods to determine peak flows may be employed if acceptable to the local floodplain administrator, the River Authority staff, and FEMA. The Rational Method is intended for local drainage only (less than 200 acres) and shall not be used for regional modeling.

3.1.1.1 Stream Gauge Analysis

Flood frequency analysis of stream gauge data should be used to determine peak flows when at least 10 years of data is available. Careful consideration must be given regarding how this data was determined, whether by direct measure or interpolation, when selecting a gauge site and utilizing the site data for flood frequency analysis. Gauges with less than 10 years of data are not sufficient for flood frequency analysis but may be used for calibration.

Watershed changes and backwater conditions will impact analysis. Watershed changes include, but are not limited to, urbanization, detention facilities, transportation infrastructure, etc. Backwater can cause the same water surface elevation to represent more than one flow during a storm event, creating a looped rating curve.

When performing a flood frequency analysis, the Pearson Type III distribution with log transformation (Log-Pearson Type III) should be used. See the publication *Guidelines for Determining Flood Flow Frequency - Bulletin 17C* (USGS, 2018) for guidance.

The USGS software, PeakFQ, and USACE's Hydrologic Engineering Center Statistical Software Package (HEC-SSP) are statistical flood frequency analysis software packages available for use. FEMA's *Statistical Models Meeting the Minimum Requirement of National Flood Insurance Program* provides a complete list of accepted statistical models.

If a flood frequency analysis is performed, the gauge numbers and source information should be documented.

3.1.1.2 Regression Equations

Regression equations may be acceptable depending on the scope of the hydrologic modeling task. They are not recommended for areas with changing land use or flood detention structures or where flood hydrographs are required for hydraulic analysis.

FEMA recommends the use of statewide regression equations. TXDOT recommends regional regression equations. If regression equations are used, document the selection process for the equation(s).

3.1.1.3 Rainfall-Runoff Models

Because many streams in the SARB have limited or no gauge data, contain sub-watersheds with flood control structures, and experience frequently changing development conditions rainfall-runoff modeling is the most common method for determining peak flows. Rainfall-runoff models apply rainfall to parameterized representations of the watershed. Rainfall can be measured or synthetic, applied at points

or applied across a grid. The parameters can be aggregated by subbasin (lumped) or gridded (distributed).

Rainfall-runoff models have been developed for most of the watersheds in Bexar County and for many watersheds in Wilson, Karnes, and Goliad Counties for the purpose of floodplain mapping. The models were developed using HEC-HMS with synthetic rainfall and lumped parameters. Documentation regarding model development is available on D2MR. The current version of this free software can be found at <http://www.hec.usace.army.mil/software/hec-hms/>. See the HEC-HMS user and technical reference manuals for capabilities and limitations.

The use of alternate modeling software is also acceptable as long as the software is included on FEMA's *Hydrologic Models Meeting the Minimum Requirement of National Flood Insurance Program* list. The only exception is HEC-1 which is not acceptable for use in the SARB. If the study is proposing to use an alternate method to the effective model method, the study must establish why the effective method is inappropriate and explain why the proposed alternate is more appropriate.

3.1.2 Level of Detail

The regional hydrologic models are statistical models based on statistical precipitation data (not historical events). As such, the precision of the estimated peak flows is limited. The following guidelines for rounding should be applied when reporting peak flows:

- If the peak flow is less than 1,000 cfs, round to the nearest whole cfs.
- If the peak flow is between 1,000 and 10,000 cfs, round to the nearest 10 cfs.
- If the peak flow is between 10,000 and 100,000 cfs, round to the nearest 100 cfs.
- If the peak flow is greater than 100,000 cfs, round to the nearest 1,000 cfs.

3.1.3 Time Step

The calculation time interval in hydrologic modeling is an important consideration and must be carefully weighed for the modeling scale. The regional nature of the proposed modeling system lends itself to the use of a longer time step than is used for more detailed studies. It is recommended that the time step for computation be a minimum of 5 minutes. This time step will provide a balance of computational efficiency and accuracy for the length of routing reaches and drainage-basin lag times that are most likely to be considered in the regional scale models.

Smaller time steps may be necessary in order to ensure computational stability of short routing reaches, to model drainage basins with a short lag time, or to update the regional models with detailed studies using smaller subbasins. If a smaller time step causes excessively long computation times, the study area may be separated from the main watershed model for use with the smaller time step. The results from the sub-model should then be incorporated into the main model.

Adjusting the time step of a hydrologic model will impact the number of steps/subreaches in the Modified Puls and Muskingum routing methods. See Section 3.6 (Routing) for guidance on routing methods.

3.2 SCENARIOS

When performing a hydrologic analysis, all of the modeling scenarios should be included in a single model file. The scenarios may include the following:

Duplicate Effective – This scenario is used when updating the software version or changing software. The topographic, land use, soil, and structural information included in the input data must match the effective hydrology model.

Corrected Effective – This scenario is used to correct errors in the effective/duplicate effective modeling configuration but does not include updates to modeling parameters due to human-caused changes since the effective model was adopted.

Existing Conditions (Pre Project) – This scenario is based on the duplicate effective or corrected effective scenario and will include non-project-related updates to reflect existing topographic, land use, soil, and structural changes to the hydrologic landscape.

Proposed Conditions (Post Project) – This scenario updates the Existing Conditions model with project-related parameters for the phase being constructed that impact land use, time of concentration, unit hydrograph or routing.

Future Conditions (Ultimate Development) – This scenario addresses planned development by applying anticipated changes in land use for the study basin (Section 1.3.2). The Future Conditions scenario shall include all future phases of a planned project.

3.3 SUBBASINS

3.3.1 Subbasin Delineation

The subbasin delineation target for DFIRM was approximately 1.5 square miles in urban areas and approximately 3 square miles in rural areas not likely to urbanize in the foreseeable future. Smaller subbasins may be required to adequately represent special features (confluences, diversion tunnels, detention facilities, etc.) within the study area. Smaller subbasins may also be required to provide boundary flow conditions at the upstream ends of hydraulic model reaches. The recommended minimum subbasin area is approximately 1 square mile, which corresponds with the lower limits for FEMA floodplain modeling. The modeler should gradually work downstream, ensuring that the rate-of-increase in computed peak discharge (from node to node) allows for a stable backwater computation. If a reservoir is included in the hydrology model, a subbasin must be delineated on the top of berm for the modeled reservoir.

When redelineating subbasin boundaries from the regional hydrology model, subbasin flows should be recalculated. In some instances, subbasin transform and/or loss parameters may also need to be recalculated to estimate the updated flows. Figure 3-1 illustrates the process of when to update the parameters for subbasin delineation.

If the updated subbasins are less than 1 square mile in size, using a proportional area weighting method to calculate flows may be considered. When using this flow interpolation method, updated subbasin delineations and proportional area weighting calculations should be provided with the analysis to support the interpolated flow values used.

Proportional area weighted calculations may be used for subbasins less than 1 square mile for LOMC submittal studies. For design analysis of flood control structures (e.g., stream crossings, channels, or detention ponds) several conditions should be met before employing area weighted calculations in lieu of re-calculating all basin parameters. If the original subbasin(s) to be subdivided are generally homogeneous in loss characteristics (e.g., CN values, Green and Ampt soil parameters, or impervious coverage percentages), the original subbasin loss parameters may be applied to the new subbasins; however, the transform parameters, (e.g., lag time and Snyder peaking coefficient C_p), should still be recalculated.

Alternatively, if the original subbasin parameters are not generally homogeneous, original subbasin loss parameters may also be used (with updates to the transform parameters) if it can be shown that recalculating the subbasin loss parameters would not result in significant changes in the overall runoff flow downstream.

If the original subbasin parameters are not generally homogenous and recalculating subbasin loss parameters may result in significant changes in flow, then both the loss parameters and the transform parameters should be recalculated for the subbasins. Refer to Section 3.4 for more information on updating Curve Number or Green and Ampt parameters.

Interpolated flows may also be used to fulfill the 5 percent rule for flow change locations in hydraulic modeling mentioned in Section 4.3.6.1. Interpolated flows at flow change locations established to fulfill the 5 percent rule may be used for long reaches within a regulatory floodplain but should not be used at hydraulic structures such as crossings, dams, or ponds.

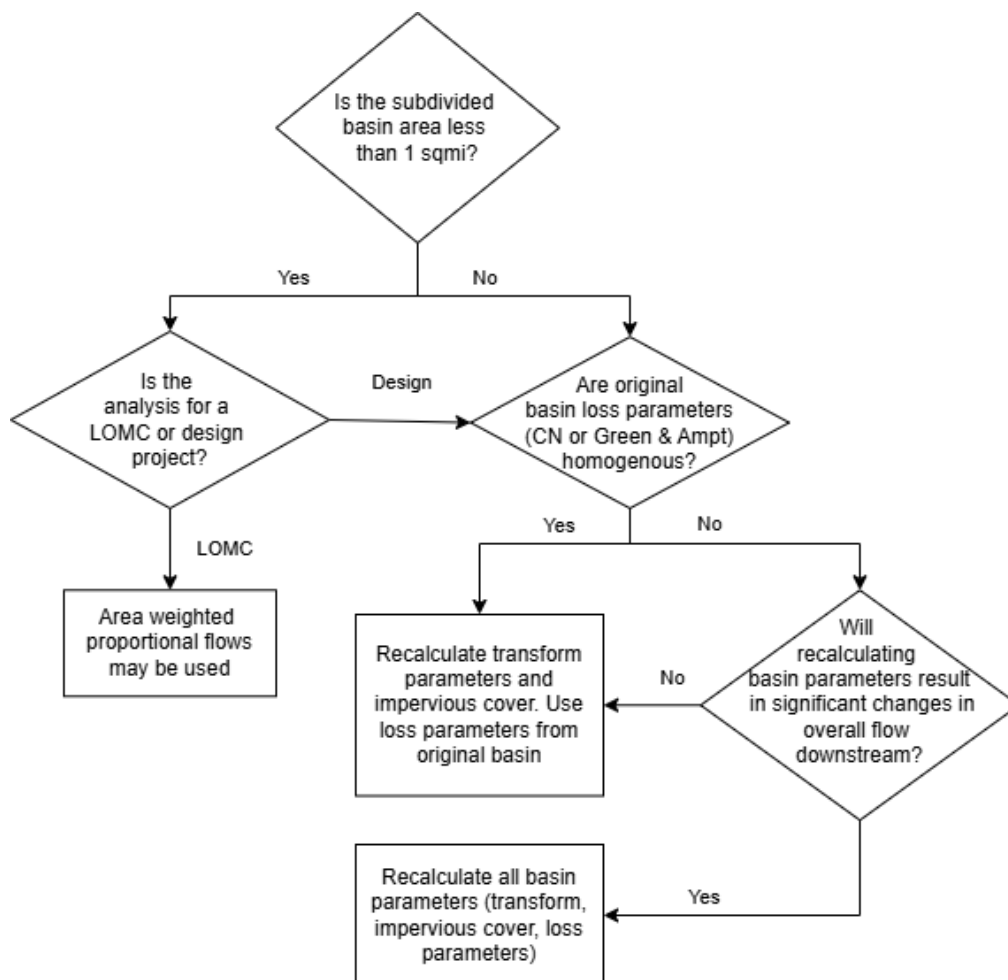


Figure 3-1 – Updating Parameters for Subbasin Delineation

3.3.2 Naming Conventions

For a comprehensive listing of current drainage basin delineations and naming, please contact the River Authority. If subbasin renaming is necessary within these models, coordination with the River Authority is requested.

Regardless of which subbasin naming convention is used, junction names should start with "J" and reach names should start with "R".

3.3.2.1 Hydrologic Unit Code Numbering

The United States Geological Survey (USGS) Hydrologic Unit Code (HUC) numbering system was adopted for Salado, Medina and Cibolo watersheds to establish unique watershed names for regional hydrology models. The HUC is defined by a series of two-number codes that define the level of detail for

the catchment under study. Thus, the 8-digit HUC for the Upper San Antonio River can be broken down as follows:

1. 12: Region: Texas-Gulf Region
2. 10: Sub-Region: Central Texas Coastal
3. 03: Account Unit: The SARB
4. 01: Cataloging Unit: The Upper San Antonio River

The SARB was originally divided into four categorized as shown in Table 3.1.

Table 3.1 - HUC Numbering System

12100301	Upper San Antonio River
12100302	Medina River
12100303	Lower San Antonio River
12100304	Cibolo Creek

The SARB HUC numbering system was increased to a 12-digit or 14-digit number in order to capture the level of detail needed for regional modeling. The 12-digit HUC was primarily used in rural areas of the basin. The 14-digit HUC was used primarily in urban areas. A 12-digit HUC is obtained by adding four digits to the original 8-digit HUC. The first two digits represent the watershed and the other two represent the sub-watershed. The 14-digit HUC is obtained by adding an additional two digits to the 12-digit HUC to represent individual drainage basins. Figure 3-2 lists the hydrologic unit breakdown for 12 and 14-digit HUCs. More information can be found in the *Federal Standards and Procedures for the National Watershed Boundary Dataset* (USGS, 2012).

14-digit	12	: Region	Texas Gulf Region	12-digit
	10	: Sub Region	Central-Texas Coastal	
	03	: Account Unit	The SARB	
	01	: Cataloging Unit	The Upper San Antonio River	
	01	: Watershed	Salado Creek	
	06	: Sub Watershed	Rosillo Creek	
	02	: Drainage Basin	Rosillo Creek Drainage Basin RC2	

Figure 3-2 - HUC Numbering Example

The HUC numbering system begins at the top of the watershed, increases in the downstream direction and ranges from 01 to 99 as shown in Figure 3-3.

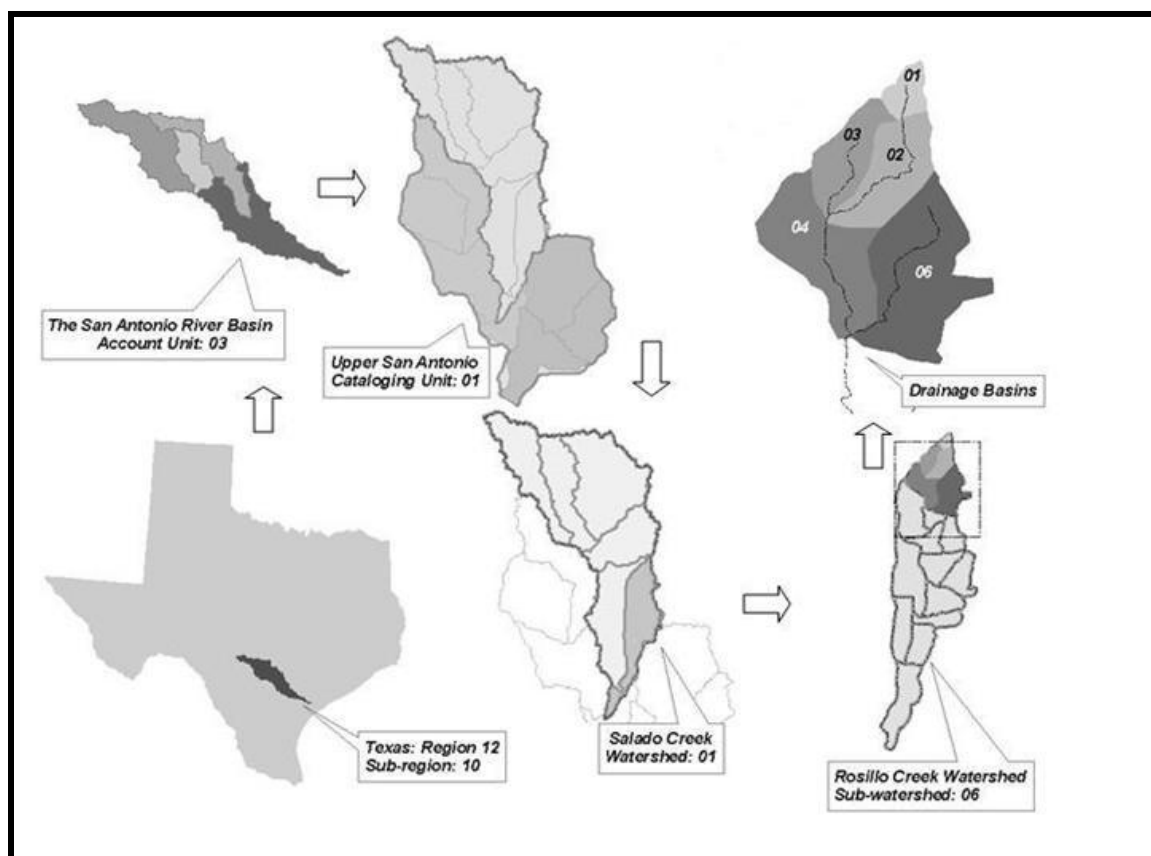


Figure 3-3 - Drainage Basin Numbering

3.3.2.2 Tributary Abbreviation Codes

Leon Creek and the Upper San Antonio River Limited Map Maintenance Program (LMMP) use a subbasin naming system based on a combination of an abbreviation of the tributary name followed by a number.

3.4 LOSS METHOD

Two primary methods of estimating losses due to interception, infiltration, storage, evaporation, and transpiration are used: SCS Curve Number (Section 3.4.1) and Green and Ampt (Section 3.4.2). In both methods, the losses are calculated for pervious surfaces while directly connected impervious surfaces are assumed to have negligible losses. Recommended impervious cover percentages, applicable for both loss methods, are discussed in Section 3.4.3.

Losses should be calculated for each scenario modeled in the hydrologic analysis including existing/pre-project conditions as well as proposed/post-project conditions. In certain instances, the engineer may also be required to model losses associated with interim conditions (e.g., for projects located in the City of Boerne where an analysis of interim construction conditions is required); in these cases, coordination with the local FPA and River Authority is recommended to determine specific loss parameters to use for the interim analysis.

3.4.1 SCS Curve Number Method

The SCS Curve Number (CN) loss model can be used to simulate surface storage and infiltration losses for single-event models. The CN method uses hydrologic soil group and type of land use to determine the

base curve number. The base curve number is then modified depending on the antecedent moisture condition.

3.4.1.1 Base Curve Number

The CN shall be based on the hydrologic soil group, antecedent moisture conditions, and natural land cover for undeveloped areas. For developed areas, a separate percent impervious cover parameter shall be used in conjunction with the CN. Composite CNs should be rounded to the nearest whole number. The CN values listed in Table 3.2 have been adopted as a representative base set for the SARB. These CNs are based on the assumption that ground cover is in good condition (>75-percent cover).

Table 3.2 - Base Curve Numbers

Cover Description	Hydrologic Condition	Curve Number for Hydrologic Soil Group			
		A	B	C	D
Open space (lawns, parks, golf courses, cemeteries, etc.)	Good	39	61	74	80
Meadow (continuous grass, protected from grazing and generally mowed for hay)		30	58	71	78
Brush (brush-weed-grass mixture with brush the major element)	Good	30	48	65	73
Woods	Good	30	55	70	77

An expanded set of CNs can be developed using the USGS National Land Use/Land Cover (NLCD) data described in Section 2.2.1. This data set uses a modified form of the Anderson Land Use classification system to define 21 land use categories. In order to be useful, each of these classifications must be associated with a CN. The recommended associations between NLCD and NRCS land use class types are listed in Table 3.3 based on NRCS class types described in the NRCS National Engineering Handbook (NEH) Part 630 Chapter 9 (NRCS, 2004).

Note: The NLCD classification for quarries/strip mines/gravel pits is not adequately covered by any category. These areas, especially in the karst portions of the watershed, will tend to hold significant amounts of water or allow for rapid infiltration rather than increased runoff. If the area is freely draining, the same CNs for transitional areas should be used. If the area tends to hold water, an appropriately low curve number should be selected, or the area should be excluded as a non-contributing or disconnected drainage area.

Table 3.3 - National Land Cover, Land Use Classifications and Corresponding NRCS Classifications

NLCD Class	NRCS Class	Hydrologic Condition
11 Open Water	NA – Assume 98 for all	
12 Perennial Ice/Snow	NA	
21 Low Intensity Residential	Open space with COSA impervious percentage for ¼ to 1 acre Res.	Good
22 High Intensity Residential	Open space with COSA impervious percentage for ½ acre Res.	Good
23 Commercial/Industrial/Transportation	Open space with COSA impervious percentage from 72–95%	Good
31 Bare Rock/Sand/Clay	Newly graded areas	
32 Quarries/Strip Mines/Gravel Pits	Special – refer to text	

	NLCD Class	NRCS Class	Hydrologic Condition
33	Transitional	Newly graded areas	
41	Deciduous Forest	Woods	Fair
42	Evergreen Forest	Woods	Fair
43	Mixed Forest	Woods	Fair
51	Shrubland	Brush	Fair
61	Orchards/Vineyards/Other	Woods – grass combination	Good
71	Grasslands/Herbaceous	Meadow	
81	Pasture/Hay	Pasture	Fair
82	Row Crops	Row crops (SR+CR)	Good
83	Small Grains	Small grain (SR+CR)	Good
84	Fallow	Fallow	Good
85	Urban/Recreational Grasses	Open Space	Good
91	Woody Wetlands	Woods	Good
92	Emergent Herbaceous Wetlands	Meadow	

3.4.1.2 Antecedent Moisture Condition

The SCS Curve Number method also requires an assumed initial moisture condition. The soil moisture standard for the SARB for most hydrology studies is Antecedent Moisture Condition II. If long-term or period-of-record simulations are being performed, a continuous moisture accounting methodology shall be used. Continuous moisture accounting is typically used to evaluate long-term flow conditions for erosion and sediment transport studies, or to more accurately simulate historical events that lasted more than one day. HEC-HMS provides several options for continuous moisture accounting. The study should fully document the reasoning for the selection of a particular method.

Adjustments can be made to reflect more appropriate antecedent moisture conditions using Table 3.4. Any adjustments to the antecedent moisture condition must be documented in the study report.

Table 3.4 - Variation of Curve Number with Antecedent Moisture Condition

CN for Condition II	CN for Condition I	CN for Condition III	CN for Condition II	CN for Condition I	CN for Condition III
100	100	100	61	41	78
99	97	100	60	40	78
98	94	99	59	39	77
97	91	99	58	38	76
96	89	99	57	37	75
95	87	98	56	36	75
94	85	98	55	35	74
93	83	98	54	34	73
92	81	97	53	33	72
91	80	97	52	32	71
90	78	96	51	31	70
89	76	96	50	31	70
88	75	95	49	30	69
87	73	95	48	29	68

CN for Condition II	CN for Condition I	CN for Condition III	CN for Condition II	CN for Condition I	CN for Condition III
86	72	94	47	28	67
85	70	94	46	27	66
84	68	93	45	26	65
83	67	93	44	25	64
82	66	92	43	25	63
81	64	92	42	24	62
80	63	91	41	23	61
79	62	91	40	22	60
78	60	90	39	21	59
77	59	89	38	21	58
76	58	89	37	20	57
75	57	88	36	19	56
74	55	88	35	18	55
73	54	87	34	18	54
72	53	86	33	17	53
71	52	86	32	16	52
70	51	85	31	16	51
69	50	84	30	15	50
68	48	84			
67	47	83	25	12	43
66	46	82	20	9	37
65	45	82	15	6	30
64	44	81	10	4	22
63	43	80	5	2	13
62	42	79	0	0	0

3.4.2 Green and Ampt Method

Infiltration losses can also be accounted using the Green and Ampt equation. The Green and Ampt model is based on Darcy's law which assumes that the vertical flow through the soil column is homogenous with uniform antecedent moisture content in the soil profile before infiltration. During infiltration, the Green and Ampt model assumes that soil above the wetting front is completely saturated. To estimate runoff using this loss method, the infiltration loss rate is subtracted from the rainfall rate for each hydrologic time step; if the infiltration loss rate exceeds the rainfall rate for a given time step, then no runoff occurs at that time.

Many of the latest regional DFIRM hydrologic models have been updated to utilize the Green and Ampt loss method over the SCS Curve Number method. Whereas the SCS Curve Number method relies on empirical data, the Green and Ampt method is a physically based model that uses measurable soil properties such as infiltration rates, soil suction, and porosity. For this reason, the Green and Ampt method can provide more accurate hydrologic model results, particularly when detailed soil data are available.

The Surface Storage Loss method is typically used in conjunction with the Green and Ampt method in the latest regional models to represent ground surface depression storage effects. This method is discussed in Section 3.4.4.

3.4.2.1 Soil Parameters

Green and Ampt soil parameters are determined from predominant soil types and soil horizons which influence the infiltration capacity within the sub-watershed. Soil parameter values may be obtained from on-site geotechnical investigation, U.S. Department of Agriculture (USDA) SSURGO data, or from established sources (i.e., correlating soil parameters with soil texture class).

The Green and Ampt infiltration loss method includes the following four soil properties estimated for each subbasin:

- Hydraulic conductivity, K (in/hr),
- Suction head (or wetting front capillary pressure, in),
- Initial water content (in/in, %), estimated as 70-percent of the *effective* porosity, and
- Saturated water content (in/in, %), taken as the *total* porosity.

The *hydrologic conductivity* of the soil is the measure of how easily water flows through the soil. Hydraulic conductivity is highly dependent on environmental conditions. While the fundamental Green and Ampt model assumes fully saturated conditions above the wetted front, actual soil conditions often deviate significantly from this assumption. As a result, it is common practice to apply the *effective hydraulic conductivity* (K_e) (the estimated effective rate of water moving through partially saturated soil) to the Green and Ampt model rather than *saturated hydraulic conductivity* (K_s) (the theoretical rate of water moving through fully saturated soil). In the absence of specific field data to estimate effective hydraulic conductivity, K_e may be estimated as $0.5 K_s$ (Bouwer, 1969).

The *suction head* or *wetting front capillary pressure* of the soil describes the forces that attract water within the void spaces of the soil column.

The *saturated water content* of the soil is the volumetric water content (the amount of water by volume in the soil) or water-holding capacity of the soil under saturated conditions, often estimated as equal to the *total* porosity of the soil.

The *initial water content* of the soil is the volumetric water content (the amount of water by volume in the soil) before infiltration. Antecedent moisture condition should be considered when specifying this parameter. In the absence of specific field data or calibrated values, initial water content may be estimated as approximately 70-percent of the *effective* porosity.

In addition to the soil properties described above, the Green and Ampt infiltration loss method uses a parameter to account for percent impervious cover. Impervious cover for existing development and zoning within SARB can be parameterized according to the guidance provided in Table 3.6. Note that HEC-HMS does not calculate losses for the portion of the sub-watershed specified as impervious cover.

Green and Ampt soil parameters can be obtained from the following sources:

- **Geotechnical Report**

A geotechnical investigation of soil properties should include soil texture, porosity, hydraulic conductivity, matric potential, or suction. The geotechnical report should be prepared by a licensed engineer with geotechnical expertise, a licensed geologist, and engineering geologist, hydrogeologist, or other licensed professional acceptable to the local jurisdictions. The report should include methods used to describe the soil properties, the laboratory results including the quality assurance methods, and field methods describing sample acquisition. The results should include adequate data from sampling sites to show onsite variability.

- **SSURGO Data**

The SSURGO dataset is available from the USDA web soil survey. The SSURGO data is specified as the default data reference. If the specific parameters are not available for the sites of interest, the soil parameters listed in Table 3.5 can be used in the model. The SSURGO dataset reports a range of values for numerous variables along with the mean value. The mean value should be used unless alternate justification is provided. The River Authority has processed a geospatial dataset from SSURGO dataset which is available to use at the River Authority's open data portal.

- **Default Values Based on Soil Texture Class**

In the absence of specific data from either geotechnical report or geospatial data from SSURGO, predominant soil type controlling infiltration capacity and published values of the relevant variables shown in soil texture class (see Table 3.5) could be used. The value should be within the observed range of values, which is usually close representative of value of the class.

Table 3.5 - Green and Ampt Infiltration Parameters for Various Soil Texture Classes (adapted from Rawls et al., 1983)

Soil Texture Class	Porosity¹ (in/in)	Effective Porosity² (in/in)	Suction (in)	Conductivity (in/hr)
Sand	0.374-0.50 (0.437)	0.354-0.480 (0.417)	0.38-9.98 (1.95)	4.64
Loamy Sand	0.363-0.506 (0.437)	0.329-0.473 (0.401)	0.53-11 (2.41)	1.18
Sandy Loam	0.351-0.555 (0.453)	0.283-0.541 (0.412)	1.05-17.90 (4.33)	0.43
Loam	0.375-0.551 (0.463)	0.334-0.534 (0.434)	0.52-23.56 (3.50)	0.13
Silt Loam	0.420-0.582 (0.501)	0.394-0.578 (0.486)	1.15-37.56 (6.57)	0.26
Sandy Clay Loam	0.332-0.464 (0.398)	0.235-0.425 (0.330)	1.74-42.52 (8.6)	0.06
Clay Loam	0.409-0.519 (0.464)	0.279-0.501 (0.390) ³	1.89-56.02 (8.22)	0.04
Silty Clay Loam	0.418-0.524 (0.471)	0.374-0.517 (0.432)	2.11-51.77 (10.75)	0.04
Sandy Clay	0.370-0.490 (0.430)	0.207-0.435 (0.321)	1.61-55.20 (9.41)	0.02
Silty Clay	0.425-0.533 (0.479)	0.334-0.512 (0.423)	2.41-54.88 (11.50)	0.02
Clay	0.427-0.523 (0.475)	0.269-0.501 (0.385)	2.52-61.61 (12.45)	0.01

¹ Total porosity may be used for *saturated water content*

² 70-percent of *effective* porosity may be used to estimate *initial water content*

³ While many sources report this value as 0.309, this value is in error and has been corrected here

3.4.2.2 Using the Green and Ampt Method

The latest regional DFIRM hydrologic models were each developed utilizing watershed-specific considerations and methods to establish Green and Ampt loss parameters. These methods are documented in the associated Hydrologic Report for each watershed. The main stem reaches of these models were calibrated to historical rain events. When working with these models, it is important to always refer to the specific watershed's report for specific methods and assumptions.

For example, in some watersheds (e.g., Salado Creek) a simple area-weighted approach was used to assign all soil parameters to specific subbasins, while in other watersheds (e.g., Upper San Antonio River) a more complex log area weighting approach was used to assign hydraulic conductivity and suction head to the subbasins. In addition, due to the complexities of watershed calibration, various methods were used to calibrate Green and Ampt parameters, such as adjusting the initial water content or hydraulic conductivity parameters (e.g., applying a 0.5 multiplier to convert K_s to K_e , applying a maximum cap on hydraulic conductivity to limit the rate of infiltration for specific areas, or applying other calibration adjustments).

When performing a study using the regional hydrologic models, the effective Green and Ampt subbasin parameters may be used to simplify the analysis if certain circumstances are met. If soil textures within the study area are homogeneous or the proposed subbasin redelineations have similar soil characteristics as the effective subbasins (e.g., when subdividing basins into smaller subbasins), Green and Ampt parameters from the effective subbasins may be applied to the revised subbasins. Any assumptions made related to Green and Ampt parameters should be documented in the study's hydrologic report.

If soil textures are non-homogeneous or the proposed subbasin redelineations require moving the overall subbasin drainage boundary, Green and Ampt parameters may need to be updated. If updating the Green and Ampt parameters is necessary, refer to the specific watershed Hydrologic Reports to verify consistency with the watershed's calibration parameters. Coordination with the River Authority is recommended prior to modifying Green and Ampt parameters to ensure consistency in approach.

When updating regional models for LOMC studies, any updates to Green and Ampt parameters should be documented and compared with existing Green and Ampt parameters. Refer to Section 1.2 (Model Updates) for additional guidance on updating regional models.

As a generalized approach, calculating loss parameters using the Green and Ampt method may include the following steps separated into two parts:

Part 1 – Calculate Green and Ampt soil properties (steps 1-4) or use existing Green and Ampt soil properties data from regional model (skip to step 5)

1. *Obtain or develop spatial soil texture data:* Soil textures may be estimated using physical soil properties data from SSURGO (% sand, % silt, % clay) and the USDA Soil Texture Triangle (USDA, 2024). SSURGO soil data may be downloaded using a variety of aggregation methods (dominant component, weighted averaging) and depths (0cm to 200cm). Consider using the weighted averaging method and 15cm (6 inch) depth to start.
2. *Estimate soil porosity, effective porosity, and suction head parameters from the identified soil texture classes:* Physical soil parameters may be derived from soil texture values using the Rawls Green and Ampt parameters (Table 3.5).
3. *Estimate hydraulic conductivity soil parameters:* Soil hydraulic conductivity values may be estimated directly from SSURGO hydraulic conductivity data. Note that SSURGO hydraulic conductivity data are provided as *saturated hydraulic conductivity* (K_s) and may need to be adjusted

to *effective hydraulic conductivity* (K_e) using a 0.5 multiplier depending on the application. Also note that SSURGO hydraulic conductivity values are expressed in units of micrometers per second and should be converted to inches per hour for HEC-HMS (1 $\mu\text{m/s}$ = 0.141732 in/hr). Like SSURGO soil texture data (discussed in step 1), hydraulic conductivity values may be downloaded from SSURGO using a variety of aggregation methods and depths. Consistent aggregation and depth methods should be used for all data obtained from SSURGO and documented in the associated hydrologic report.

4. *Estimate initial water content as 70-percent of the effective porosity and apply any other calibration adjustments or caps:* For specific information on assumptions or methods used for each watershed, refer to the individual watershed Hydrologic Reports.

Part 2 – Update subbasins soil parameters in HEC-HMS based on updated soil properties data (steps 1-4) or based on existing Green and Ampt soil properties data from regional model

5. *Use area-weighted averaging or area-weighted log averaging to assign soil parameters to individual subbasins:* Specific area-weighted averaging approaches (including log averaging formulas) are discussed in the watershed-specific Hydrologic Reports. For additional details on the log averaging method, refer to Section 2.6.2.3 from the Leon Creek Watershed Hydrology Report (2022).
6. *Assign subbasin parameters to HEC-HMS:* Based on area-weighted subbasin values, populate HEC-HMS Green and Ampt values for hydraulic conductivity, saturated water content (total porosity), initial water content (70-percent effective porosity), and suction head.

3.4.3 Impervious Cover

Recommended impervious cover percentages for existing development and zoning within the SARB are listed in Table 3.6.

Table 3.6 - Percent Impervious Cover by Land Use

	Land Use Category	Zoning District	Average Percent Impervious Cover
Residential	1/8-acre Residential Lots, or Garden or Townhouse apartments	R-4, R-5, RM-4, RM-5; TND/TOD Use Patterns	65-85%
	1/4-acre Residential Lots	R-6, RM-6	38%
	1/3-acre Residential Lots	R-15	30%
	1/2-acre Residential Lots	R-20	25%
	1-acre Residential Lots	RP, RE	20%
	Industrial	L, I-1, I-2	72-85%
	Business or Commercial	NC, O, C	85-95%
	Densely developed (apartments)	MF	65-85%
	Streets, Roads, and Parking Areas		98%

3.4.4 Surface Storage Loss

In addition to using the Green and Ampt loss method, many of the latest regional DFIRM hydrologic models utilize the Surface Storage Loss method to represent the ground surface where water may accumulate in surface depression storage. Recommended surface storage losses by land use type within the SARB are listed in Table 3.7.

Alternatively, surface storage losses may be estimated based on specific site conditions or measurements provided that the losses are described and justified in the associated model documentation. Examples of acceptable sources for alternate surface storage losses include percolation tests, soil moisture sensors, or assessments of vegetation and ground cover.

Table 3.7: Surface Storage Losses by Land Use

	NLCD Class	NRCS TR-55 Class	Surface Loss (Range)	Surface Loss (Recommended)
11	Open Water	NA – Assume 98 for all	NA	NA
12	Perennial Ice/Snow	NA	NA	NA
21	Low Intensity Residential	Open space with COSA impervious percentage for ¼ to 1 acre Res.	0.1 - 0.5	0.30
22	High Intensity Residential	Open space with COSA impervious percentage for ⅛ acre Res.	0.1 - 0.5	0.15
23	Commercial/Industrial/Transportation	Open space with COSA impervious percentage from 72–95%	0.1 - 0.5	0.05
31	Bare Rock/Sand/Clay	Newly graded areas	0.05 - 0.15	0.15
32	Quarries/Strip Mines/Gravel Pits	Special – refer to text	0.5 - 1.5	1.00
33	Transitional	Newly graded areas	0.05 - 0.15	0.15
41	Deciduous Forest	Woods	0.2 - 0.5	0.40
42	Evergreen Forest	Woods	0.2 - 0.5	0.40
43	Mixed Forest	Woods	0.2 - 0.5	0.40
51	Shrubland	Brush	0.25 - 0.5	0.35
61	Orchards/Vineyards/Other	Woods – grass combination	0.2 - 0.4	0.35
71	Grasslands/Herbaceous	Meadow	0.2 - 0.4	0.30
81	Pasture/Hay	Pasture	0.2 - 0.4	0.30
82	Row Crops	Row crops (SR+CR)	0.15 - 1.5	0.50
83	Small Grains	Small grain (SR+CR)	0.15 - 1.5	0.50
84	Fallow	Fallow	0.2 - 0.4	0.30
85	Urban/Recreational Grasses	Open Space	0.2 - 0.4	0.30
91	Woody Wetlands	Woods	0.2 - 0.6	0.35
92	Emergent Herbaceous Wetlands	Meadow	0.2 - 0.6	0.35

3.5 HYDROGRAPH

3.5.1 Method Selection

The selection of a unit hydrograph method is based on topography, purpose, and software limitations. The SARB contains a wide range of topographic conditions from the steep slopes of the Edwards Plateau to the moderate slopes in the San Antonio area to the flat slopes toward the Gulf Coast, and the unit hydrograph method needs to be able to account for this variation.

The COSA UDC recommends the use of the SCS dimensionless unit hydrograph², but the implementation of the SCS unit hydrograph method in HEC-HMS has limitations. In HEC-HMS, the SCS methodology includes a hard-wired shape factor of 484. The factor has been known to vary from about 600 in steep terrain to 100 in very flat terrain (NRCS, 2007a). While the SCS method may be appropriate for the moderately sloping areas that cover San Antonio and Bexar County south of the Edwards Plateau, it is not as applicable for the steeply sloping areas in the upper portions of the basin or the flat areas toward the coast. Other methods such as the Snyder and Modified Clark unit hydrograph methods

² The UDC allows for the use of other methods with approval.

provide additional variables that allow the user to adjust the shape of the hydrograph to more closely match observed conditions.

Because of the limitations of the SCS dimensionless unit hydrograph, the Snyder unit hydrograph is the recommended unit hydrograph methodology for consistency across the entire SARB. In areas appropriate for the application of the SCS method (moderate topography) either Snyder or SCS methodology may be used. Existing models may retain the SCS methodology.

The Modified Clark (ModClark) methodology may also be used in cases where a gridded representation of the watershed is appropriate and has been defined. The ModClark method is typically used in conjunction with radar data.

3.5.2 SCS Unit Hydrograph

The SCS Unit Hydrograph method requires two parameters, lag time (T_{lag}) and a shape factor (also called a peak rate factor). T_{lag} for a drainage basin is typically calculated as a fraction of the time of concentration (T_c) for the basin. An estimate of T_{lag} as 60-percent of T_c based on studies by the NRCS is commonly used (used in COSA UDC). Various studies have estimated T_{lag} as 50–75-percent of T_c . A value of 0.6 T_c should be used as the initial estimate for T_{lag} . However, this variation should be considered during calibration of models. The estimated value for T_{lag} should be rounded to the nearest minute.

For ungauged watersheds, T_c can be estimated using the method described in the NRCS NEH Part 630 Chapter 15 (NRCS, 2010). Design hydrology models may also use the Seelye chart to estimate the overland flow portion of T_c as outlined in the COSA UDC.

The NRCS methodology separates the movement of water through the drainage basin into three components. These components and the resources for calculation of each are listed below.

1. Overland (sheet) flow (OF)
 - a. Acceptable range = $5 < OF < 20$ minutes
 - b. Use Manning's kinematic solution (Equation 15-8) (NRCS, 2010)

$$T_t = \frac{0.007 (nL)^{0.8}}{(P_2)^{0.5} S^{0.4}}$$

Where:

T_t = travel time (hr),
 n = Manning's roughness coefficient for sheet flow (Table 3.8),
 L = flow length (ft),
 P_2 = 2-year, 24-hour rainfall (in), and
 S = slope of land surface (ft/ft).

- c. Maximum length = 300 feet with a most-likely length of 100 feet when used in overland flow computations for unpaved areas. (Thomas, 1986)

Table 3.8 - Manning's Roughness Coefficients for Sheet Flow (Depth ≤ 0.1 ft)

Surface Description	n
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover ≥ 20%	0.17
Grass:	
Short-grass prairie	0.15
Dense grasses	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:	
Light underbrush	0.40
Dense underbrush	0.80

Source: NRCS NEH Part 630 Chapter 15 Table 15-1 (NRCS, 2010)

2. Shallow concentrated flow

- a. Use Manning's equation for defined swales, bar ditches, and street sections

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

Where:

V = velocity,
n = Manning's roughness coefficient for open channel flow,
R = hydraulic radius,
S = channel slope, and
1.49 is a conversion factor for English units.

- b. Use Figure 15-4 from NRCS NEH Part 630 Chapter 15 where the geometric section is not defined (NRCS, 2010)

3. Channel flow

- a. Existing computer model where available
- b. Manning's equation otherwise

The peak rate factor can range from 100 in flat, swampy areas to 600 in steep areas. In HEC-HMS, the peak rate factor is fixed at a value of 484.

3.5.3 Snyder Unit Hydrograph

The Snyder unit hydrograph requires two parameters, lag time and peaking coefficient (similar function to the "484" shape factor coefficient in SCS unit hydrograph) to define the shape of the hydrograph.

The Snyder lag time (T_{lag}) is calculated using the following equation:

$$T_{lag} = CC_t \left(\frac{LL_c}{\sqrt{S}} \right)^N$$

Where:

C_t = basin coefficient,
 L = length of the main stream from the outlet to the divide (miles),
 L_c = length along the main stream from the outlet to a point nearest the watershed centroid (miles),
 C = a conversion constant (0.75 for SI and 1.00 for foot-pound system),
 S = overall slope of longest watercourse from point of concentration to the boundary of drainage basin (vertical feet/horizontal miles), and
 N = an exponent, commonly taken as 0.33.

The estimated value for T_{lag} should be rounded to the nearest minute. The parameter C_t is best found via calibration, as it is not a physically based parameter. This parameter has been found to vary from 0.4 in mountainous areas to 8.0 along the Gulf of Mexico. The C_t parameter is based on percent developed:

$$C_t = 1.4224e^{-0.0088x}$$

x = the percentage of development = $1.75 * \text{Percent Impervious}$ (**not to exceed 100%**)

Since the C_t parameter is considered a calibration parameter, specific C_t values and methods may vary across different watersheds. For example, the Salado Creek watershed regional model uses an alternate method for estimating the C_t parameter. When comparing transform parameters with regional models, refer to the model's associated hydrology report for any specific calibration adjustments.

The peaking coefficient (C_p) is also generally determined based on calibration to measured events. In a Technical Memorandum prepared for Bexar Regional Watershed Management titled "Snyder Unit Hydrograph Parameter Guidelines, San Antonio River Basin Regional Watershed Modeling System," (PBS&J, 2005) the following initial values are recommended:

Subbasins on or above the Balcones Escarpment – 0.71
 Subbasins between the Balcones Escarpment and the confluence of the San Antonio River and Cibolo Creek – 0.55
 Subbasins below the confluence of the San Antonio River and Cibolo Creek – 0.45

The values used in the regional hydrologic models have been adjusted from these initial values based on calibration. Exhibits in Appendix F show the values for Snyder Peaking Coefficient used in the regional hydrologic models.

3.5.4 Modified Clark

The Modified Clark (ModClark) model is used with gridded meteorological data and distributed parameter models. The Modified Clark Unit Hydrograph model is a linear quasi-distributed unit hydrograph which explicitly accounts for variations in travel time throughout the watershed by accounting for translation and storage. It requires two input parameters for each grid cell: distance to watershed outlet and cell area. Additional information can be found in *Runoff Simulation Using Radar Rainfall Data* (USACE HEC, 1996) and the *HEC-HMS Technical Reference Manual* (USACE HEC, 2000). To use the ModClark model within HEC-HMS the grid cell parameters are stored in a *.mod file and can be developed using HEC-GeoHMS or other GIS software tool. It is based on a HRAP grid or standard hydrologic grid (SHG) and should be developed in a projection system that is based on metric units. The grid cell parameters should be

developed in the same coordinate system as other parameter and gridded rainfall (or RADAR) datasets that are to be used as inputs to the model. In addition, the time of concentration and storage coefficient must be specified for each subbasin.

3.6 ROUTING

The channel routing methodology selected for HEC-HMS modeling will be based on the availability of hydraulic models within the watershed. If hydraulic models are available, the Modified Puls routing technique should be applied. For channels without available hydraulic models, the selection of the routing method will be based on the anticipated effects of storage. If overbank/channel storage is not significant, the Muskingum-Cunge eight-point cross-section method should be used. If the effects of overbank/channel storage are anticipated to be significant, the Muskingum routing method should be used.

The order of preference for routing procedures to be used in the SARB models is as follows:

1. Modified Puls based on hydraulic modeling,
2. Muskingum-Cunge with 8-point cross section,
3. Muskingum

The number of routing steps within a routing reach must be set for each of these methods in order to ensure the computational stability of the model. The number of steps should be based on an estimate of the flow or travel time as described in the HEC-HMS manuals and the model time-step.

3.6.1 Modified Puls

The storage-discharge relationship for Modified Puls routing is to be based on the finalized hydraulic models for the watershed. These models must be configured for execution over a range of flow that brackets the minimum (0 cfs) and maximum flows (greater of 0.2-percent AC existing or 1-percent AC ultimate conditions). At least eight different flow values (not including 0) distributed proportionally over this range should be used to develop the routing data. The resulting storage-discharge curves should be checked for discontinuities prior to use in the hydrologic models. Such discontinuities or sharp breaks in the curve are likely to cause computational instabilities in the hydrologic model.

3.6.2 Muskingum Cunge

The Muskingum-Cunge 8-point cross section should be based on the best available topographic or as-built plan data for the subject reach. Manning's n-values should be determined in a similar fashion to those for hydraulic models.

3.6.3 Muskingum

The Muskingum routing method requires the input of two parameters. These are the travel time through the reach (K) and a dimensionless weighting factor (X) related to the character of the channel.

The travel time can be estimated as $K = L/V_w$. The USACE Engineering Manual EM 1110-2-1417 (USACE, 1994) suggests estimating the flood wave velocity (V_w) as 1.33–1.67 times the average velocity, which may be estimated using Manning's equation and representative cross-section geometric information.

The X coefficient is related to the type of channel and condition of flow in the channel. Table 3.9 provides guidelines for this coefficient. These are intended to serve as a starting point and may be modified during calibration.

Table 3.9 - Guidelines for Initial Values of the Muskingum X Coefficient

Type of Channel		X Coefficient
Natural with Overbank Flow	Flat	0.1
	Moderate	0.2
	Steep	0.3
Regular Grass Lined Ditch		0.4
Culverts or Concrete Channels		0.5

3.7 CHANNEL LOSSES

Within the Edwards Aquifer Recharge Zone, seepage of surface flow into the aquifer results in streamflow losses. Streamflow losses for the Edwards Aquifer Recharge Zone are described in the 2007 USGS study of Upper Cibolo Creek (Ockerman, 2007) as well as the 2002 USGS Water-Resources Investigations Report 02-4241 (Ockerman, 2002). Based on the relationship shown in Figure 6 from the 2002 USGS study (calculated for Salado Creek for the October 1998 storm event), streamflow losses can be estimated as:

$$\text{Streamflow loss per stream mile (cfs)} = 0.062 \times \text{streamflow in cfs} + 8.3$$

In HEC-HMS, channel losses are entered in the Reach data using the “Constant” loss/gain method which has two parameters, Flow Rate (cfs) and Fraction where:

$$\text{Flow Rate (cfs)} = 8.3 \times \text{reach length in miles} \quad \text{Fraction} = 0.062 \times \text{reach length in miles}$$

The fraction and flow rate factors should be adjusted with calibration. In Cibolo, fraction was reduced to $0.01 \times \text{reach length in miles}$.

Calibrated channel loss assumptions are discussed in the watershed-specific Hydrologic Reports in the following sections:

- Cibolo Creek: Table 5 (Section 2.4.2), Appendix B
- Leon Creek: n/a
- Lower Medina River Tributaries: n/a
- Salado Creek: Figure 3-4 (Section 3.3.6), Appendix A.9
- San Geronimo Creek: Table 7 (Section 2.9.3), Figure 10 (Section 2.9.3), Section 3.5.4
- Upper San Antonio River: Table 10 (Section 2.12.4), Figure 15 (Section 2.12.4), Table 29 (Section 3.4.5)

3.8 STORAGE AND DIVERSIONS

Before modeling a detention facility, its storage capacity and significance to the watershed should be considered. Regional models should include major storage and diversion structures such as regional detention basins, NRCS flood control structures, and larger dams and reservoirs. Development-level detention ponds used for storm water quality or quantity management should be included in the design hydrology model with impacts to the regional model reflected in the impervious cover and/or lag time parameters.

Storage and diversions should be modeled based on the best available information from as-built design plans or field surveys. Some facilities may need to be analyzed independently of the overall hydrologic model. The results of the external analysis shall then be input into the regional model. Such facilities would include Medina Dam and certain off-channel regional detention facilities that have complex inflow and outflow relationships. The standard methodology available within HEC-HMS should be used for all other structures and diversions. Only volume above the normal pool elevation of a facility should be included in storage estimates. Results for NRCS structures should be compared with existing NRCS SITES models where available. The River Authority should be consulted prior to altering the storage/diversion curves for existing tunnels or the River Authority's NRCS dams.

For ponds/stock ponds located within the modeled watershed where data is not available, the engineer should use engineering judgment to determine if evaluating the pond in the hydrologic model will have a significant impact on the hydrologic model results. The engineer should consider drainage area to pond versus overall stream drainage area, size of pond, and location of pond (online or offline). Documentation for pond evaluation decisions made should be included in the project documentation and coordinated with local FPAs and the River Authority. Inclusion of ponds in hydrologic models will be at the discretion of the local FPAs and the River Authority. Any FEMA submittals including ponds must support the pond modeling with survey or as-built information.

For steady state hydraulic models, the WSEs results at upstream and downstream bounding cross sections of reservoirs should match output stage results from HEC-HMS for the corresponding storm event(s). Upstream bounding cross sections should be modeled with defined known WSEs based on HEC-HMS reservoir stage results. Downstream bounding cross sections should be modeled as a flow change location using HEC-HMS reservoir flow discharge results. Refer to Section 4.3.5 for guidance on the hydraulic model.

When incorporating dams or reservoirs into unsteady hydraulic models, it is preferred that output results from the hydrologic model be used to establish boundary conditions at the structure (rather than using storage rating curves in the hydraulic model). Hydrologic model output results should be linked directly as an input in the hydraulic model when possible (like in the case of HEC-RAS and HEC-HMS sharing DSS files). WSE results should be checked for consistency between hydrologic and hydraulic models.

3.9 RAINFALL

Rainfall is the driving factor behind all processes simulated in H&H models. Hydrologic analysis for floodplain mapping is based around design storms rather than specific historic events. A reasonable hypothetical representation of rainfall is a key factor in the production of good design storm simulations. These aspects include the spatial and temporal distributions of the rainfall, the duration of the rainfall, and the total amount of rainfall.

3.9.1 Duration

A 24-hour design storm is sufficient for most watershed level evaluations. However, modeling watersheds with longer response times such as the entire SARB will require a longer simulation time and possibly a longer design storm. All models should be checked to ensure that the peak flow has passed the mouth of the basin prior to the end of the simulation (the outfall hydrograph should have reached the falling limb near the baseflow as shown in Figure 3-4).

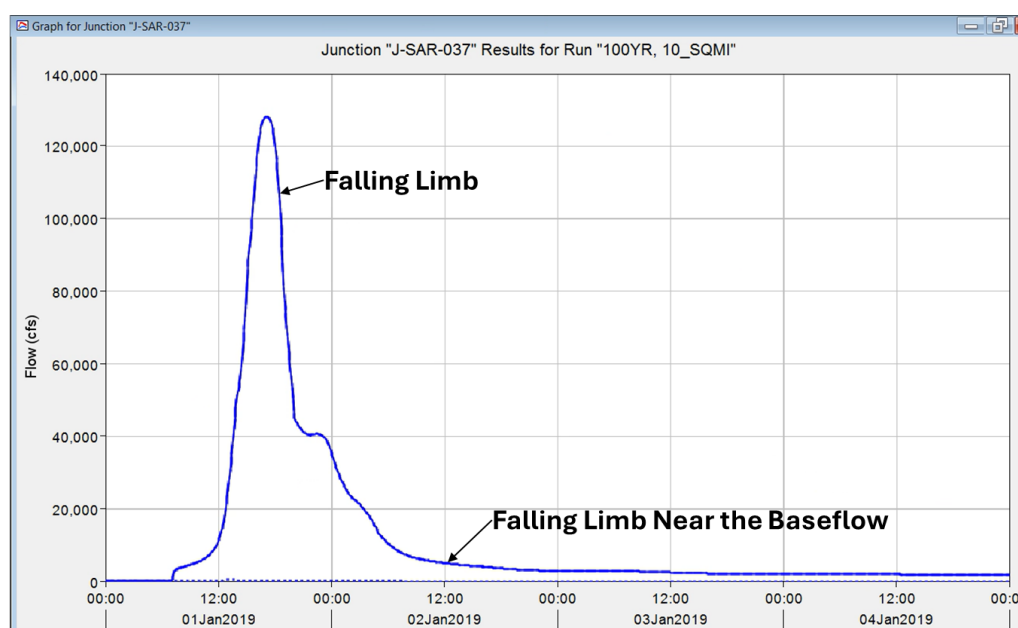


Figure 3-4 – Example Hydrograph for Checking Model Duration

3.9.2 Precipitation

The total rainfall volume is selected from the tables in Section 2.5 based on design storm event, storm duration, and county in which the study is located.

3.9.3 Distribution

The temporal distribution of the design storm rainfall is assigned in HEC-HMS using the USER HYETOGRAPH storm simulation type. The DFIRM hyetographs are stored in a Hydrologic Engineering Center Data Storage System (HEC-DSS) file which is available with the HEC-HMS model provided by the River Authority. They were developed to address the temporal and spatial distribution of rainfall within the SARB.

The nature of the spatial distribution for synthetic rainfall will be determined by the scale of the modeling to be performed.

For Watershed Scale Modeling, an approach of using uniformly distributed rainfall across the entire watershed is used. The use of point rainfall totals is appropriate for areas less than or equal to 10 square

miles. All modeling for drainage areas greater than 10 square miles in the SARB will apply areal reduction factors (ARF) to the rainfall to represent equivalent average values over the contributing watershed area.

The DFIRM hyetographs in the HEC-DSS file were developed to facilitate the adoption of updated rainfall values from NOAA Atlas 14 Precipitation-Frequency Atlas of the United States Volume 11: Texas (Perica, et al., 2018). A set of areal reduction curves were developed for the SARB and can be found in Appendix C. For a detailed description of the process please refer to Technical Paper 2, Appendix D. These areal reduction curves were used in Bexar, Karnes, and Wilson Counties. Goliad County used areal reduction factors developed for coastal areas as noted in the Hydrology TSDN for Goliad County. The hyetographs have to be stored and referenced outside of HEC-HMS because the integrated method of computing areal reduction within the software is based on the reduction curves in TP-40. The TP-40 areal reduction factors will produce excessive runoff compared to the runoff resulting from application of the new areal reduction factors.

If River Basin Scale Modeling is performed, references such as Hydrometeorological Report (HMR 52) (NOAA, 1982) for guidance on developing ellipsoidal distribution of rainfall.

For information about the approach used in a particular regional model, refer to its Hydrology Report.

3.10 BASEFLOW AND INFLOW HYDROGRAPHS

In some cases, a model will need to account for flow from areas beyond the model limits or from baseflow.

The DFIRM regional hydrology models did not account for baseflow. Baseflow in the modeled stream segments was less than 5-percent of the peak flow for the design storm events. If a study is performed in which the baseflow is greater than 5-percent of the peak flow for the design storm event, the baseflow should be included in the analysis, and the source of baseflow data should be documented.

Flow from areas beyond the model limits is represented using inflow hydrographs. Several of the regional hydrology models include inflow hydrographs from adjoining regional models as documented in the hydrology Reports. Inflow hydrographs may also be used in design hydrology models to represent upstream areas truncated from the regional models. When inflow hydrographs are used, methodologies and simulation timing in the source model and the analysis model must be consistent.

3.11 DOCUMENTATION

3.11.1 Model Metadata

Each model developed for the SARB must be thoroughly documented. This documentation should be in a form similar to the metadata required for geospatial data. It will include documentation of the source of the model, the spatial extents, the methodologies used in the modeling, and any significant differences from previous models in the area. A summary of the additional information to be included in the modeling metadata is listed below.

1. Common Information
 - a. Responsible Engineer
 - b. Major Watershed
 - c. Brief Description (<250 characters)
 - d. Description of any unusual features
 - e. Purpose of study
 - f. Extents of modeling
 - g. General differences from preceding models
 - h. Date completed by engineer
 - i. Date of jurisdictional review
 - j. Date of River Authority review
 - k. File Name
2. Hydrology Specific Information
 - a. Design storm description
 - b. Unit hydrograph methodology description
 - c. Routing methodology description
 - d. Source model for Modified Puls routing data
 - e. Source of soils data
 - f. Source of land use data
 - g. Development condition

These items should serve as the minimum requirements for documentation of the models in the SARB. This information will be stored in a linked database and maintained by the River Authority so that it can be accessed to determine the source of data within the modeling geodatabase. This is in addition to the documentation of the modeling process required by FEMA for update of FIRMs.

3.11.2 Geospatial Documentation

If the following elements are created or modified:

- Subbasins,
- Reaches,
- Junctions, or
- Locations of modeled structures,
- Land Use

The supporting geospatial files are requested by the River Authority in the format outlined in Appendix E. The data should only be submitted for the elements being modified within the study area.

4 HYDRAULIC MODELING

Hydraulic modeling is used both to calculate the extent of the floodplain and to evaluate the impact of structures and any changes made to a channel. It is important that the hydraulic modeling methodology used for the SARB be consistent for all models. Any elements of the hydraulic modeling application that are not specifically listed below (ineffective flow areas, blocked obstructions, levees, etc.) are to be based on generally accepted engineering principles.

4.1 APPROACH

4.1.1 Model Type

Most regional hydraulic models used for floodplain mapping in the SARB are one-dimensional steady state riverine models. Unsteady state and two-dimensional modeling techniques are acceptable; however, if modifications to a DFIRM hydraulic model include changing model type, supporting documentation must include an explanation of why the existing modeling approach is inappropriate.

4.1.1.1 One-dimensional steady state

One-dimensional steady state models represent flow in one direction: from upstream to downstream. Hydrologic input does not vary with time. This is the primary model type used for floodplain mapping and analysis in the SARB.

4.1.1.2 One-dimensional unsteady state

Often the hydraulics of a study reach is complex. When flow timing and lateral flow connections are vital to the accuracy of a model, an unsteady model may be considered. Unsteady flow modeling is appropriate when the assumptions for steady flow modeling are no longer valid. Examples include reaches with overland flow combined with storm sewer flow, analysis of manhole overflow in conjunction with street flooding, interaction at stream junctions where timing of peak flow from one stream affects the hydraulics of another, or modeling spills from one stream to another. Engineering studies intending to use unsteady models must document why the traditional steady-state approach cannot be used and why an unsteady model is required to comply with the scope of the study. The documentation should include the reasoning behind software selection.

Applications of the unsteady flow option within HEC-RAS should be based on the guidelines in the HEC-RAS manuals and in the UNET model (original basis for the unsteady flow capabilities in HEC-RAS) documentation.

4.1.1.3 Two-dimensional unsteady state

Two-dimensional models can simulate flow in multiple directions and are therefore useful for modeling flat areas and complex drainage patterns. Two-dimensional model surfaces can be developed to represent the entire study area or can be linked to one-dimensional model features.

4.1.2 Software Selection

The hydraulic modeling software selected as the standard for use in DFIRM models was the USACE Hydraulic Engineering Center – River Analysis System (HEC-RAS). Software updates and patches occur regularly, as this software continues to be developed. More recent versions are considered acceptable. Current version of this free software can be found at <http://www.hec.usace.army.mil/software/hecras/>. For capabilities and limitations please refer to the HEC-RAS User's Manual and Hydraulic Reference Manual.

The use of other hydraulic modeling software is also acceptable depending on the modeling task, as long as the software is included on FEMA's *Hydraulic Models Meeting the Minimum Requirement of National Flood Insurance Program*.

4.2 SCENARIOS

When preparing a model for submittal, the appropriate scenarios should be included in a single model file. The scenarios may include but are not limited to:

- Duplicate Effective – This scenario is used when updating the software version or changing software. The topographic and structural information included in the input data must match the effective hydraulic model.
- Corrected Effective – This scenario is used to correct errors in the effective/duplicate effective modeling configuration and natural changes in topography but does not include updates to modeling parameters due to human-caused structures since the effective model was adopted.
- Existing Conditions (Pre Project) – This scenario is based on the duplicate effective or corrected effective scenario and will include non-project-related updates to reflect existing topographic, and structural changes to the hydrologic landscape.
- Proposed Conditions/Post Project – This scenario updates the Existing Conditions model with project-related parameters for the phase being constructed that impact channel or overbank geometry.
- Future Conditions (Ultimate Development) – This scenario applies the future conditions hydrology to the proposed conditions hydraulic geometry to calculate future conditions water surface elevations for regulatory purposes.

Model naming conventions should follow the scenarios listed above and only include appropriate files. For 1D steady state models, models should include a consolidated plan file for each applicable model scenario, each containing all relevant storm events for the scenario. For 1D and 2D unsteady state models, since plan files can only contain data for a single storm event, separate plan files are required for each storm event per scenario.

4.3 OPEN CHANNEL RIVERINE MODELS

4.3.1 Cross-Sections

In accordance with FEMA's *Guidance for Flood Risk Analysis and Mapping – Hydraulics: One Dimensional Analysis*, "cross-sections must be placed perpendicular to flood flow and extend beyond the most extreme event modeled... on either side of the stream." The US Army Corps of Engineers *Engineering Manual 1110-2-1416 Appendix D* (USACE, 1993) and the USGS publication *Computation of Water-Surface Profiles in Open Channels* (Davidian, 1984) provide additional guidance for good practice in the placement of cross sections.

4.3.1.1 Spacing

The primary issue in hydraulic modeling is often the density of cross sections within the modeled stream reach. The appropriate model cross-section density will be determined based on the level of development and the location of critical structures within the reach. For all modeling within cities and other urbanized portions (including areas of projected urbanization) the maximum modeling cross-section spacing will not exceed 500 feet. For areas of Bexar County and other portions of the SARB that are likely to remain predominantly rural, the maximum modeling cross section spacing will not exceed 1,000 feet. The

modeling cross-section spacing should be reduced as necessary to model significant features and steeper channel slopes. Cross-sections in the vicinity of bridges, culverts, and weirs should be located as described in the HEC-RAS manuals.

When establishing cross sections for meandering streams in rural areas the 1,000-foot minimum spacing can be difficult to maintain. In cases of severe stream meandering, especially with significant overbank flooding, consider modeling the stream using a 2D modeling approach rather than 1D for improved floodplain accuracy. When developing cross sections using a 1D modeling approach, consider the overall representative direction of flow for the primary storm event(s) being modeled. This representative flow direction may not necessarily be consistent with the topography of the meandering channel centerline in cases where flow exceeds the main channel and travels along the direction of the overall floodplain. In these instances, adjust the cross sections, bank stations, and downstream reach lengths so that the stream centerline, left of bank (LOB), and right of bank (ROB) flow paths are consistent with the representative direction of flow rather than the lowest topography. If the minimum spacing of 1,000 feet still cannot be maintained with these considerations, provide justification with the modeling documentation for the larger spacing of cross sections.

4.3.1.2 Reach Lengths

During the development of a model, reach lengths are assigned to the channel, left overbank, and right overbank areas of each cross section. These reach lengths represent varying distances along different parts of the cross section to the next downstream cross section and affect the volume within the modeled reach. While channel reach lengths should typically be measured along the thalweg line at the center of the channel, left and right overbank reach lengths should be measured along the estimated flow path through the center of mass of the overbank flow. For channels with relatively consistent paths and cross section geometries, the values for the three reach lengths will often be similar. However, for channels with varying paths or geometries (such as at river bends or where the channel meanders and overbanks are straight), the three reach lengths will differ significantly.

When modifying an existing model to add or remove a cross section or structure, the downstream reach lengths should be checked, and the volume within the model should be checked. The overall stream length for the study area should not change without justification and should correspond to the distance between and stationing of cross sections.

When a modeled stream is located near a confluence with another stream, the modeled stream centerline should always be drawn to snap to the location of the confluence stream (this should be done prior to calculating reach lengths).

4.3.1.3 Naming Convention and Stationing

Cross sections should be named numerically based on the cumulative distance (in feet) between the cross section and the downstream end of the reach (as determined from channel reach lengths). For example, a cross section that is 1,000 ft from the downstream end of the reach based on channel reach lengths would be named "1000". Cross sections numbers should be rounded to the nearest whole number.

Cross section elevation stationing should start at zero (0) at the left side of the cross section and increase along the cross section from left to right. Cross sections should not be defined using negative stationing (where station zero is in the middle of the channel).

4.3.1.4 Head Losses

Roughness coefficients should be assigned based on the roughness or frictional resistance of the channel or surface to the flow of water. Roughness values will typically be higher in overbanks with

similar vegetation to the adjacent channel due to shallower depths in the overbank. Table 4.1 and Table 4.2 list guidelines for the base Manning's n-values to be used for channels and overbank areas, respectively. Roughness values may also be adjusted slightly based on calibration procedures for the model. The actual values used for modeling should be adjusted based on site reconnaissance or aerial photography.

Table 4.1 – Manning's Roughness Coefficients (Channel)

Channel Description	Manning's n-values		
	Average	Minimum	Maximum
Concrete Lined Channel	0.015	0.010	0.020
Grass Lined Channel with regular maintenance	0.035	0.030	0.040
Gravel or Outcropping Stone Channel with limited vegetation	0.045	0.040	0.050
Grass Lined Channel without recent maintenance	0.050	0.045	0.055
Vegetated Channel with trees, little or no underbrush	0.055	0.050	0.060
Natural Channel with trees, moderate underbrush	0.075	0.070	0.080
Natural Channel with trees, dense underbrush	0.090	0.085	0.095
Natural Channel with dense trees, and dense underbrush	0.100	0.100	0.100 (or higher with justification)

Table 4.2 – Manning's Roughness Coefficients (Overbank)

Bexar Current Land Use	Overbank Description	Manning's n-values		
		Recommended	Minimum	Maximum
Commercial	Commercial and Business	0.085	0.065	0.12
Extraction Mining	Quarries/Strip Mines/Gravel Pits	0.065	0.04	0.08
Industrial	Light Industry	0.07	0.07	0.09
	Heavy Industry	0.1	0.09	0.12
Open Space Cultivated	Planted Cultivated Herbaceous	0.06	0.04	0.06
	Urban Parks	0.05	0.05	0.07
	Pasture, No Tree Coverage, 0%	0.045	0.04	0.055
Open Space Urban	Cement/Asphalt	0.02	0.015	0.024
	Grass (Good Condition)	0.035	0.03	0.045
Residential Dispersed	Residential, Dispersed (> 1 ac)	0.06	0.05	0.08
Residential High Density	Residential, High Density (1/8 ac)	0.1	0.09	0.12
Residential Low Density	Residential, Low Density (1/2 acre)	0.08	0.07	0.1

Bexar Current Land Use	Overbank Description	Manning's n-values		
		Recommended	Minimum	Maximum
Residential Medium Density	Single Family Residential (1/4 acre)	0.09	0.08	0.1
Residential Multi Family	Multi-Family Residential	0.1	0.1	0.12
Services Utilities	Services Utilities	0.07	0.055	0.085
Transportation	Transportation (ROW)	0.035	0.03	0.045
Undeveloped Brush	Pasture, Moderate Tree Coverage, 15%-50%	0.07	0.065	0.08
	Pasture, Dense Tree Coverage, >50%	0.09	0.085	0.1
	Forested, dense tree coverage and undergrowth	0.1	0.1	0.12
Undeveloped Meadow	Bare Rock/Sand/Clay	0.03	0.025	0.04
	Grass (Good Condition)	0.035	0.035	0.045
	Pasture, No Tree Coverage, 0%	0.05	0.04	0.055
	Pasture, Isolated Tree Coverage, <15%	0.055	0.05	0.065
Water	Open Water	0.04	0.025	0.05

Manning's n-values for situations not covered in the table shall be based on generally accepted engineering principles. The USGS publication, *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains* (Arcement & Schneider, 1989) may be used as a guide for the selection of roughness coefficients within the ranges listed in Table 4.1.

Manning's n-values may be developed from a number of sources. Several modeling interfaces provide options for the initial estimation of n-values from land use. Roughness coefficients derived in this manner should be regarded as an initial estimate and should be verified by evaluations in the field, or from evaluation of aerial photography.

Eddy losses (contraction and expansion coefficients) and energy losses through bridges are addressed in Section 2.2.5 of *Guidance for Flood Risk Analysis and Mapping – Hydraulics: One Dimensional Analysis*.

4.3.1.5 Ineffective Flow Areas and Blocked Obstructions

Techniques for modeling non-conveyance areas are discussed in the HEC-RAS manual and Section 2.2.4 of *Guidance for Flood Risk Analysis and Mapping – Hydraulics: One Dimensional Analysis*. Special care should be taken when producing floodplain maps where these techniques have been used to ensure water surface profiles carried into ineffective areas are not mapped erroneously.

Ineffective flow areas represent areas within a cross section that do not contribute to channel conveyance, typically due to zero or negligible velocity. Ineffective flow areas should be set upstream and downstream of all structure crossings using a method consistent with the HEC-RAS Hydraulic Reference Manual Chapter 5 Section *Defining Ineffective Flow Areas*. When defining an ineffective flow area unrelated to a structure crossing, a description should be added within the cross-section description field for justification.

Ineffective flow areas can be either “permanent” or “non-permanent” – “permanent” ineffective flow areas remain zero-conveyance areas when overtopped, while “non-permanent” flow areas are considered

conveyance areas when overtopped. Ineffective flow areas should be set to “non-permanent” by default. If it is determined that ineffective flow areas should be set to permanent, justification must be provided.

Ineffective flow areas upstream of structures should be set to the overtopping elevation, while ineffective flow areas downstream of structures should be set between the overtopping elevation and the top of opening (as shown in Figure 4-1). If a flood profile is active (above the ineffective flow area elevation) on the upstream side of a structure, it should be active on the downstream side as well.

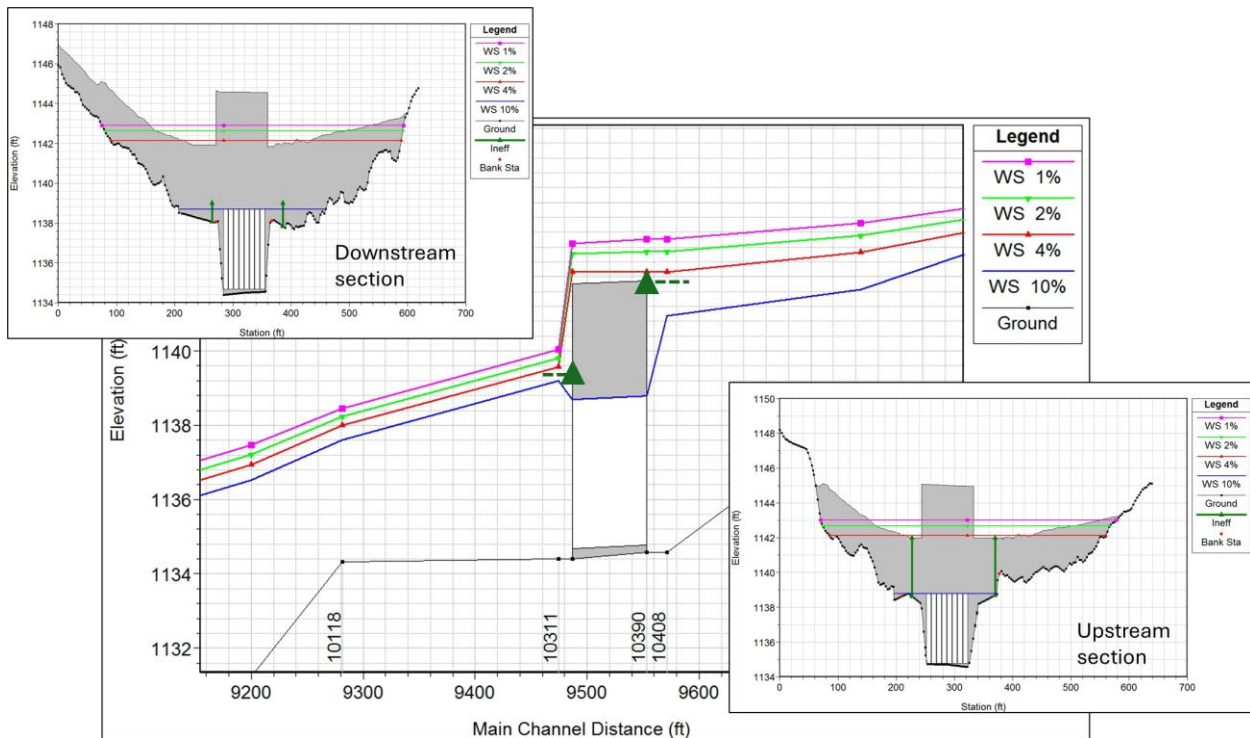


Figure 4-1 – Example ineffective flow area placement for structure in floodplain

Blocked obstructions represent areas within a cross section that should not have conveyance due to a physical obstruction within the floodplain that is not represented in the terrain. In a 1D steady state model, blocked obstructions exclude an area from both channel conveyance and depth calculations, while ineffective flow areas exclude an area from the channel conveyance calculations only. In a 1D unsteady state model, blocked obstructions also exclude an area from the storage calculation, while ineffective flow areas are still factored into storage channel storage calculations. Because of this, blocked obstructions should be used to represent physical obstructions in the channel or to exclude specific areas of the channel from depth and storage calculations.

4.3.2 Structures

The guidelines outlined in the HEC-RAS Hydraulic Reference Manual should be followed to the extent possible for all structures. This includes the proper location of cross sections upstream and downstream of the structure. Structures should be modeled based on the most reasonable available computation procedure within HEC-RAS (in-line weirs, culverts, or bridges). Certain types of culverts, weirs and inline dams may need to be modeled as bridges. This should be addressed on a case-by-case basis.

4.3.2.1 Stilling Basins and Energy Dissipator Structures

Where reaches include stilling basins, baffle blocks, or other energy dissipator structures intended to create a hydraulic jump, the user should determine if these structures are significant to the modeling results of the storm events of interest for the analysis.

In the case where it is determined that these structures are significant to the results, the model should be updated to simulate the localized effects of the structure so that the model results show the hydraulic jump these structures would create. This can be done by updating the location and number of cross sections around and through the structures to capture changes in velocity, topography, and roughness. Increased Manning's n roughness values may be used inside of the stilling basin or around baffle blocks to simulate the increased energy losses.

Please refer to the "Drop Structures" section of the HEC-RAS Hydraulic Reference Manual for more guidance.

4.3.2.2 Bridges and Culverts

The HEC-RAS bridge modeling routines provide multiple computation options for low flow and two computational options for high flow. The single most appropriate high and low flow options should be selected for the final version of the model.

4.3.2.3 Other Inline Structures

There are a number of stream reaches in the SARB with inline weirs/dams. The HEC-RAS Hydraulic Reference Manual provides guidance for selecting modeling options. The flow area in the cross sections upstream of the weirs should account for the normal pool of the structure.

4.3.2.4 Dams and Stock Ponds

If a dam is located within the extents of the hydraulic model, it should be modeled as an inline structure. Dams should be based on best available information from dam regulator and surveyed if possible, per Section 2.4 (Infrastructure Data). Stock ponds are usually located on private property. Because of this, accessing information regarding ponds is typically not possible. If a pond is located on public property, and therefore accessible, field verification should be performed.

Dams and Stock ponds should be represented with an inline structure on the embankment aligned perpendicular to flow. For Dams, flow and water surface elevations at the Dam should be calculated in the hydrologic model rather than the hydraulic model (refer to Section 4.3.5 for setting flows and water surface elevations at Dams). For Dams, the hydraulic model is also used to evaluate mapping extents and water surface elevation profiles for the stream. Refer to Figure 4-2 for an example hydraulic layout at a Dam.

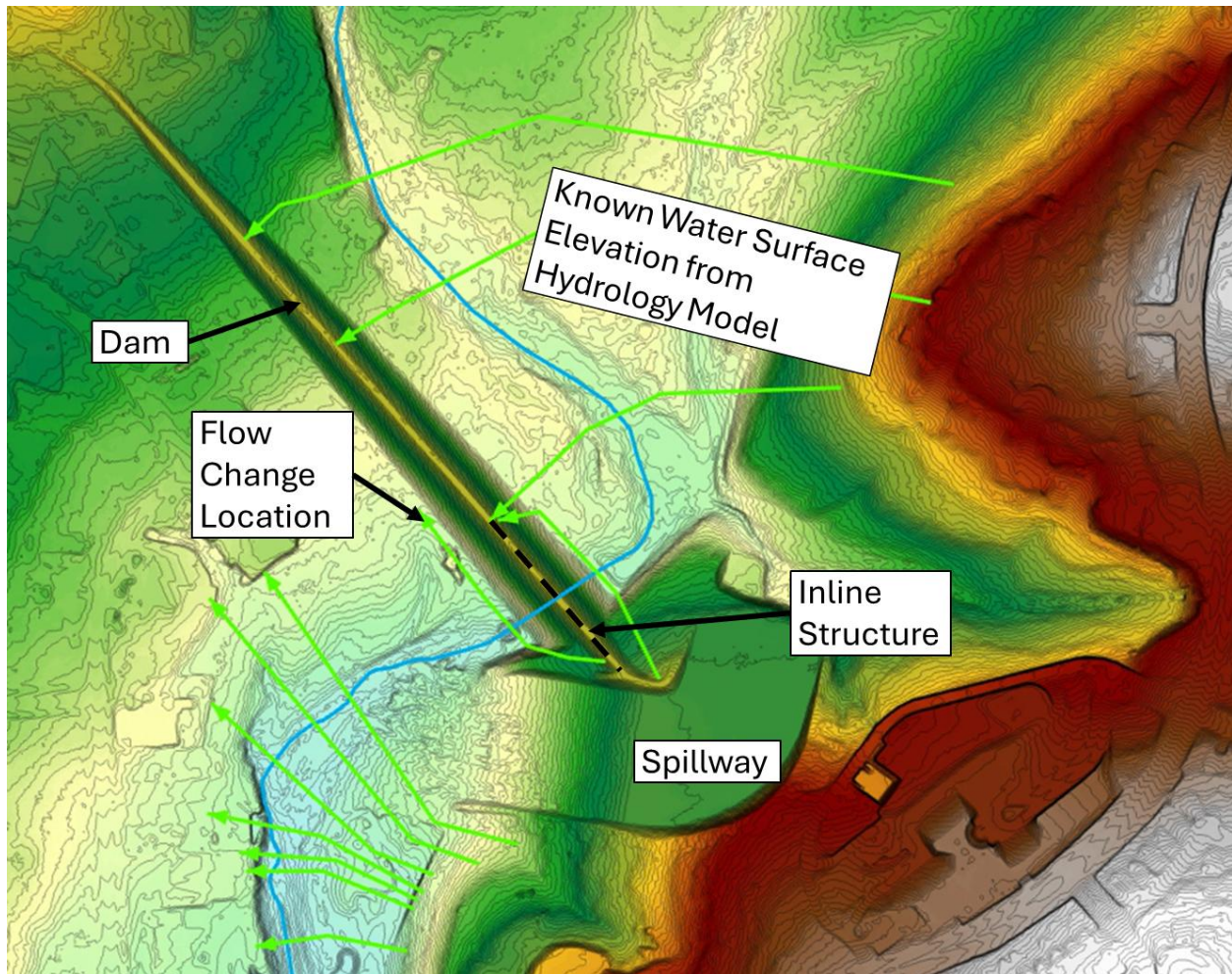


Figure 4-2 – Dam Example

For Stock ponds, the hydraulic model is used to evaluate water surface elevations and flows. When modeling stock ponds, the stream centerline should follow the overall stream direction, typically cutting through inline structure (Stock Pond #1 in Figure 4-3). If the channel elevation at the toe of the embankment is equal to the elevation at the spillway, the stream centerline should follow the spillway (Stock Pond #2 in Figure 4-3). If the embankment elevation is 4 feet or less, then a cross-section approach may be considered with the cross section capturing the impediment to flow rather than an inline structure. In the case of Stock Pond #2 in Figure 4-3, the embankment is less than 4 feet so a cross-section representing the flow impediment feature would be acceptable instead of an inline structure. If a cross section is used in place of an inline structure, bounding sections just upstream and downstream of the embankment should still be used.

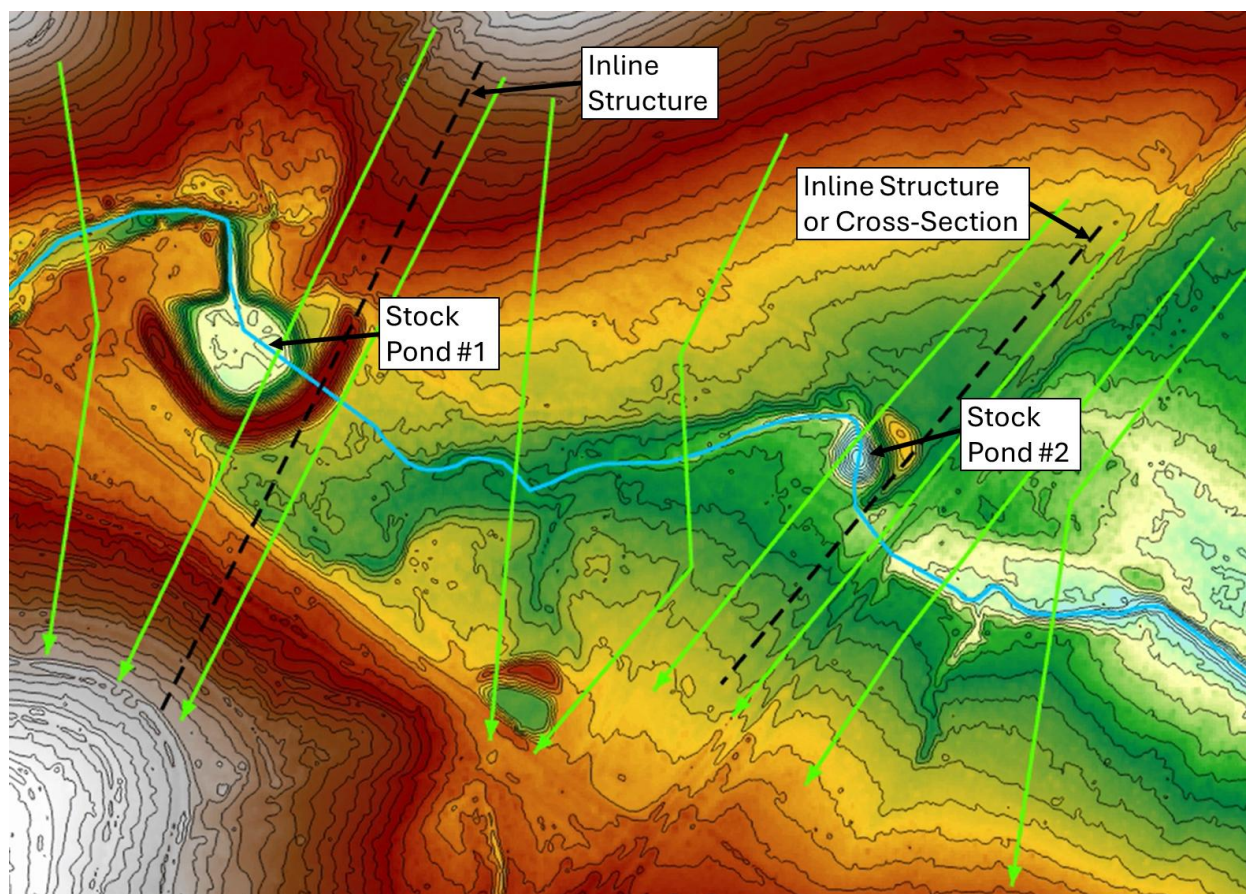


Figure 4-3 – Stock Pond Examples

4.3.2.5 Levees and Lateral Structures

Lateral structures can be used to track flow leaving the main stream at a spill location or into an offline detention pond.

When using this feature in a steady state model, the flow balance between the flow leaving and the flow continuing downstream may be determined using the Optimization feature as described in the HEC-RAS Hydraulic Reference Manual. **Please note:** the flow adjustment should then be entered as a flow change in the hydrologic input data (HEC-RAS flow file), at which point the Optimization feature should no longer be used.

When using this feature in an unsteady state model, flow optimization should be enabled.

Please refer to Section 4.6.1 (Floodways with Lateral Structures) for details on defining a floodway when a lateral structure is present.

4.3.3 Storage

Storage in hydraulic models can be divided into two types – reservoir storage and channel storage. It is handled differently between steady and unsteady state models.

In steady state models, reservoir storage is related to lateral and inline structures (above). Offline reservoirs are connected to the main channel using one of various lateral connection options as described in the HEC-RAS manual. Flow leaving the channel is optimized by balancing the water surface

elevations between the reservoir and the channel when the optimization feature is used. Inline reservoir storage is not accounted for in a steady state hydraulic model. Cross-section information needs to account for any impacts to flow area caused by the dam, and the hydrology model should be used to estimate impacts to peak flows.

While steady state models do not account for channel storage impacts to peak flows, they can be used to develop the storage-discharge relationships used for the Modified Puls hydrologic routing method (Section 3.6.1). For the DFIRM hydrology models where Modified Puls was used, reach definitions (upstream and downstream cross sections), flow tables, and routing step calculations are provided in the hydrology Reports for each regional hydrology model. The flow tables were developed using a range of flows from zero to the greater of the 0.2-percent AC existing or 1-percent AC ultimate peak flows. The tables include a sufficient number of increments to estimate a storage rating curve for each routing reach. The number of steps for each reach was calculated by dividing the reach length in feet by the product of the average velocity in feet per second and the hydrologic model time step in minutes. If a project impacts the channel geometry for a routing reach, the storage-discharge function and parameters need to be updated to reflect the changes.

4.3.4 River Reaches and Junctions

The river reach should follow the primary flowpath for the 1-percent AC storm event, usually the stream centerline. FEMA's Guidance for Flood Risk Analysis and Mapping – Profile Baseline Guidance addresses how to handle when the 1-percent AC storm event follows a different flowpath than the stream centerline (profile baseline), split flow, and diverted flow.

Where multiple reaches are combined at a junction, analysis options are described in the HEC-RAS manual. Flow optimization should be enabled at junctions for unsteady models.

4.3.5 Boundary Conditions

Boundary condition requirements depend on whether the model is steady or unsteady, whether the model limits are mid-stream or at a confluence, and the likelihood of coincident peaks at a confluence. Normal depth is the most commonly appropriate downstream boundary condition for steady flow models for floodplain mapping purposes. The normal depth boundary condition is commonly used for tributaries entering into larger controlling streams, where the floodplain extents near the confluence are primarily determined by the larger stream's backwater conditions. This follows FEMA guidance for confluences outlined in the *Guidance for Flood Risk Analysis and Mapping, Hydraulics: One-Dimensional Analysis* (FEMA, 2016). In addition, the FEMA guidance stipulates that the use of normal depth may not be appropriate in coincident peak situations and lists the following criteria to identify a coincident peak situation.

- The ratio of the drainage areas lies between 0.6 and 1.4.
- The arrival times of flood peaks are similar for the two combining watersheds.
- The likelihood of both watersheds being covered by the storm being modeled is high

When the downstream condition is not a confluence but connecting to a downstream floodplain, the normal depth boundary condition will still be required to meet WSE vertical tie-in requirements. This may require the modeler to adjust the boundary condition slope or tie-in location.

For cases outside of floodplain mapping, other types of boundary conditions should be considered if a downstream controlling backwater is expected. For steady-state modeling, known water surface elevations are acceptable boundary conditions at flood control gates and reservoirs whose water surface elevations are controlled by an outlet structure. Reservoirs should be modeled in a hydrologic model with water surface elevations for the dam pulled from the hydrologic model into the hydraulic model. For reservoirs located at the downstream end of the hydraulic model, a known water surface elevation at the dam should be used as the model's downstream boundary condition. For reservoirs located in the middle

of the model, known water surface elevations should be set for the cross sections upstream of the control structure, while known outflows should be set for the cross sections downstream of the control structure. Known water surface elevations may also be set at confluences of streams to evaluate backwater effects.

For unsteady models, stage-flow rating curves or time-flow rating curves are acceptable. If normal depth is used for unsteady models, the downstream boundary should be set well outside the study area and away from hydraulic structures such as bridges, culverts, or dams, which could impede conveyance.

4.3.6 Hydrologic Input

4.3.6.1 Steady State

When modifying the hydrologic input in a regional hydraulic model, it is generally recommended to maintain the existing flow change locations. However, modifications are acceptable with justification.

Flows used in the hydraulic model are generally the peak flows extracted from elements of the hydrology model for a given storm. The hydrology model provides flows at the downstream end of headwater basins, at junctions, and at the downstream end of routing reaches (e.g., modified puls reaches). In general, the flows used within the HEC-RAS model should either be extracted from a headwater basin, from the downstream end of a routing reach, or from a reservoir feature, unless a major tributary brings flow into a reach.

Flow change locations should be located at the end of reach segments and at confluences or at other significant inflow locations. If the peak flow increases more than 5-percent between flow change locations other than confluences or significant inflow locations, interpolated flows should be added to provide a smoother transition. The interpolation method may be weighted based on contributing drainage area or reach length depending on the nature of the drainage area between flow change locations. Refer to Section 3.3.1 for more information on interpolated flows. When modeling streams across larger watershed areas, the engineer may coordinate with the River Authority and local FPA to assess the need to increase the 5-percent peak flow change tolerance. If the flow calculation point falls on a road, the flow should be entered at the approach cross section to the bridge rather than between the actual bridge sections.

For headwater basins or if the flow change occurs at a bridge, the peak flow input will be entered at one-third the channel distance upstream from the computation flow point as shown in Figure 4-4. This location is the farthest upstream that a hydraulic model should extend into a headwater basin.

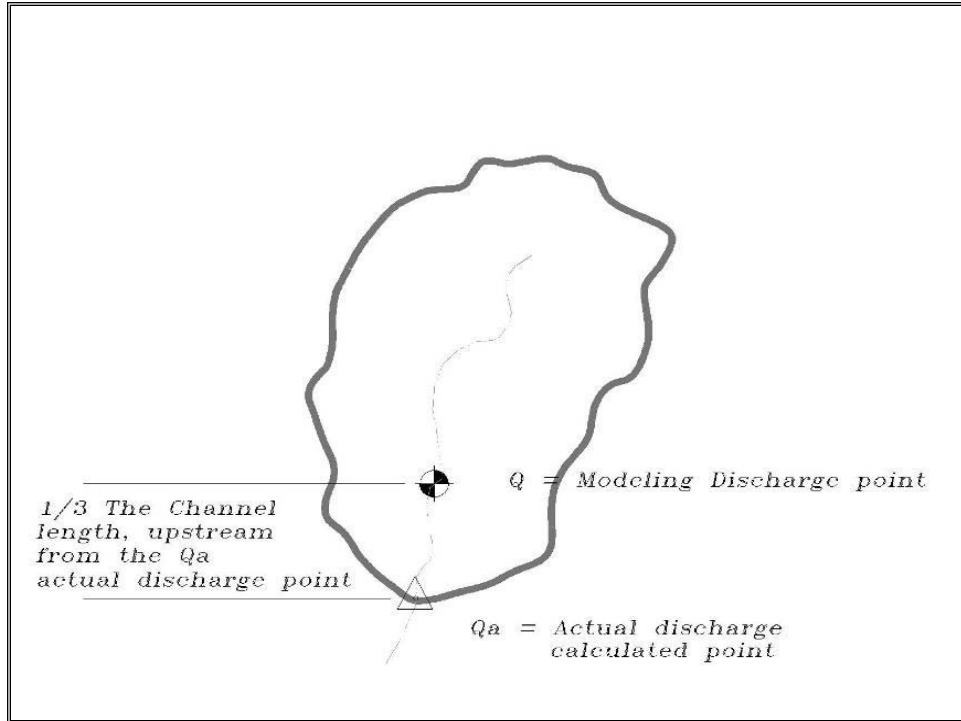


Figure 4-4 – Headwater Basin Modeling Discharge Point

If a reservoir is found to be hydrologically significant to the watershed per Section 3.8 (Storage and Diversions), flow data for the hydraulic model must be pulled from the hydrology model. If a reservoir located within the extents of the hydraulic model is included in the hydrology model, a known water surface elevation should be pulled from the hydrology model and entered in the Flow Data as a set internal change in WS, and a flow entered at the downstream end of the reservoir.

4.3.6.2 Unsteady State

Routing

Unsteady 1D and 2D hydraulic models directly account for storage routing and thus do not require routing reaches within independent HEC-HMS models to generate cumulative peak flows. Runoff hydrographs from HEC-HMS models will be input to HEC-RAS unsteady models, which account directly for storage volume using the fully dynamic St. Venant equations. Input runoff hydrographs may be individual subbasin or junction inflow hydrographs depending on the unsteady hydraulic model areas (basin headwaters, mid-reach, etc.). HEC-RAS and SWMM unsteady flow hydraulic modeling use variations of the fully dynamic St. Venant equations. Any deviations from these standards should be explained in the model documentation.

Although reach routing is explicitly modeled within an unsteady hydraulic model, independent watershed HEC-HMS models shall be maintained to produce similar routing reach conditions and consistent peak flow results. In order to maintain consistency between peak flows generated at HEC-HMS junctions and HEC-RAS unsteady flows, a comparison of peak flow, time to peak and change in storage shall be developed and documented. Differences in hydrologic results which results in measurable water surface elevations differences greater than 0.5' shall be investigated and discussed with reviewing agencies. Steps shall be made to minimize deviation in results or provide justification for differences.

Common causes for differences in hydrologic results include reach storage volume function misrepresentation, application of subbasin inflow hydrographs in unsteady models (point or uniform flow type) or location of inflow hydrographs. Greater consistency may be achieved by adjusting inflow hydrograph locations within the unsteady hydraulic model, or by changing the means of assigning the catchment hydrographs, perhaps from a uniform distribution over the length of a stream segment to a point inflow at a given location along that segment, or vice versa. For example, a catchment hydrograph that is uniformly distributed over a reach in HEC-RAS may be assigned instead as a point inflow at the upstream or downstream end of a reach in HEC-HMS. Conversely, a catchment flow hydrograph input at point location within a reach segment will also capture different attenuation/storage effects from a catchment signed at the upstream or downstream end of the reach in HEC-HMS. In making decisions as to where and how hydrographs are input, account for actual inflow locations, local geography, topography and terrain, and possible backwater effects from roads, railways, etc. Sensitivity analysis and final assumptions should be explained in the model documentation.

Inflow Hydrographs

- Modeling entire stream with Unsteady Flow Modeling with watershed area less than 10 Square-miles:

Flow at the most upstream cross section should correspond to the downstream drainage point of the most upstream subbasin as seen in Figure 4-5. The most upstream subbasin should have a target area of 1.0 square-mile. All intervening flows through the watershed need to be modeled as lateral inflow hydrographs using a flow change location, at the basin outlet of the subbasin / tributary. When modeling the entire stream with unsteady modeling using HEC-RAS, the modeler is required to submit a HEC-HMS model with routing included in the model that corresponds to Modified-Plus routing methodology.

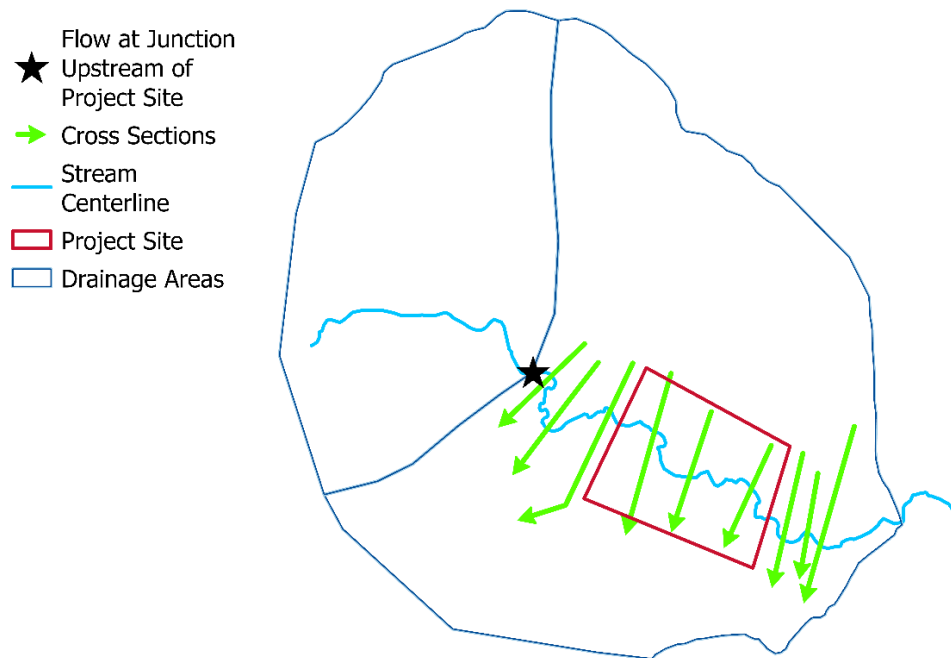


Figure 4-5 – Upstream Cross Section Flow Junction Example

- Modeling entire stream with Unsteady Flow Modeling with watershed area greater than 10 Square-miles:

The principles of areal reduction cannot be coupled with unsteady flow modeling of entire stream for watersheds greater than 10 square-miles. Therefore, unsteady flow modeling for modeling entire stream that has a watershed area of more than 10 square-miles is complicated by areal reduction. Coordination with the review agency in advance is requested.

4.3.7 Additional Considerations for Unsteady 1D Riverine Models

4.3.7.1 Stabilizing Unsteady Models

Unsteady flow models generally require more detail to setup compared to steady flow models because they require an iterative convergence of energy loss calculations at every time step. In some instances, instabilities can occur in unsteady flow models due to overly or insufficiently detailed input data.

When addressing stability issues in unsteady state models, ensure smooth geometry transitions between cross sections, both horizontally and vertically, avoiding abrupt changes in elevation or excessively steep slopes. Interpolated cross sections in HEC-RAS may be used to identify areas where additional cross sections would improve model stability. However, these interpolated cross sections should be replaced with modeled cross sections based on survey or terrain data before finalizing the model.

To further improve model stability, review hydraulic property curves for discontinuity and verify that bridge hydraulic parameter curves (HTab curves) are smooth with minimal discontinuity. Additionally, examine the model's initial conditions and consider setting initial conditions from a previous run file.

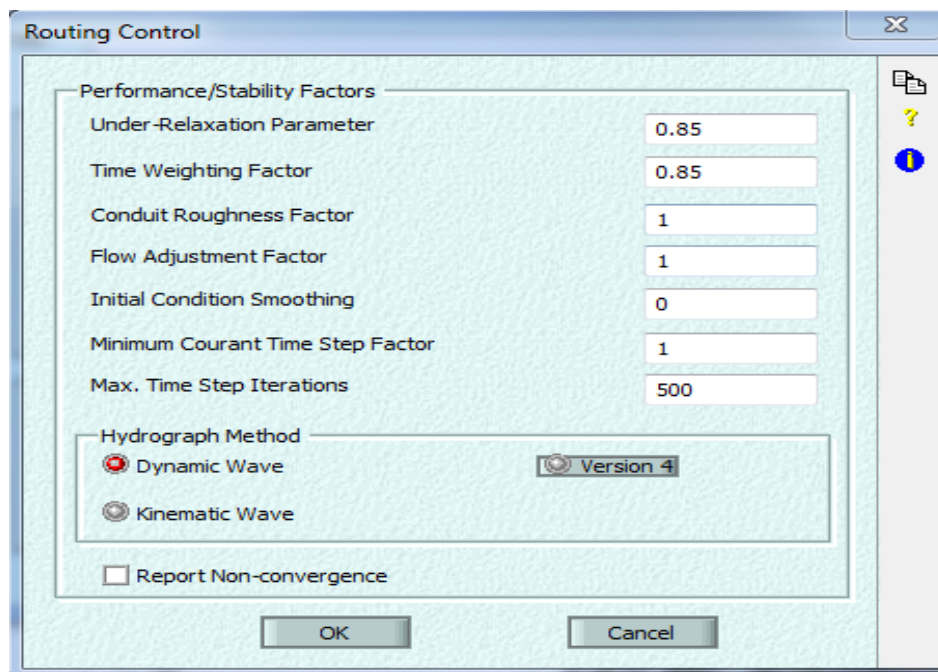
For additional guidance on improving unsteady flow model stability, refer to the HEC-RAS User's Manual section on "Performing a 1D Unsteady Flow Analysis" for more guidance.

4.3.7.2 Computation Time Step and Display Interval

Maximum computation timestep interval and hydrograph output interval for unsteady flow modeling in HEC-RAS should be 5 minutes. The time interval needs to be lowered as needed to achieve model stability. For 1D XPSWMM models, the user should start with a computation time interval of 60 seconds and reduce the time interval as necessary to achieve an acceptable continuity error of less than 5 percent.

4.3.7.3 Tolerances

The following are default tolerances used in XPSWMM:



The Routing Control dialog box contains the following settings:

Parameter	Value
Under-Relaxation Parameter	0.85
Time Weighting Factor	0.85
Conduit Roughness Factor	1
Flow Adjustment Factor	1
Initial Condition Smoothing	0
Minimum Courant Time Step Factor	1
Max. Time Step Iterations	500

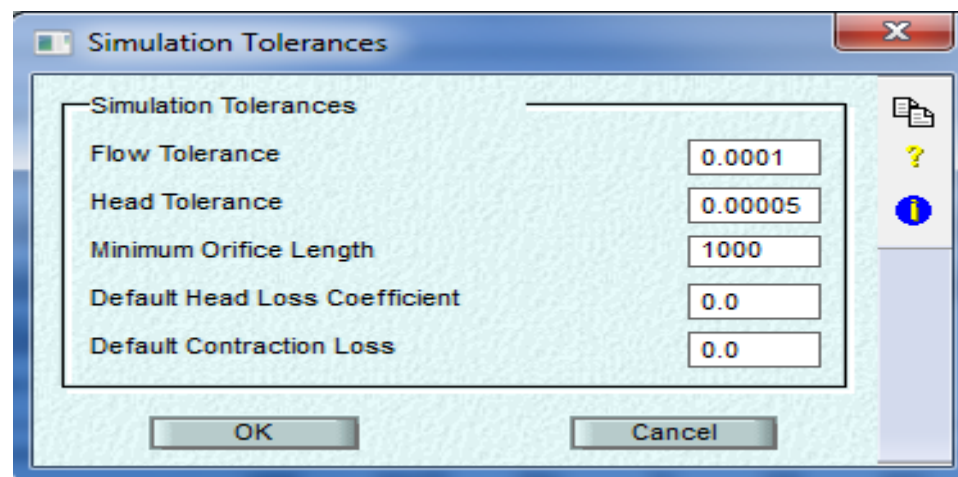
Hydrograph Method:

- ☒ Dynamic Wave
- ☐ Kinematic Wave

Version 4

☐ Report Non-convergence

Buttons: OK, Cancel



The Simulation Tolerances dialog box contains the following settings:

Parameter	Value
Flow Tolerance	0.0001
Head Tolerance	0.00005
Minimum Orifice Length	1000
Default Head Loss Coefficient	0.0
Default Contraction Loss	0.0

Buttons: OK, Cancel

The following are default tolerances in HEC-RAS:

HEC-RAS Unsteady Computation Options and Tolerances

Geometry Preprocessor Options

- ☒ Convert Energy Method Bridges to Cross Sections with Lids

Family of Rating Curves for Internal Boundaries

- ☒ Use existing internal boundary tables when possible.
- ☐ Recompute at all internal boundaries

Unsteady Flow Options

Theta [implicit weighting factor] (0.6-1.0):	1
Theta for warm up [implicit weighting factor] (0.6-1.0):	1
Water surface calculation tolerance (ft):	0.02
Storage Area elevation tolerance (ft):	0.05
Flow calculation tolerance [optional] (cfs):	
Maximum number of iterations (0-40):	20
Number of warm up time steps (0-200):	0
Time step during warm up period (hrs):	0
Minimum time step for time slicing (hrs):	0
Maximum number of time slices:	20
Lateral Structure flow stability factor (1.0-3.0):	2
Inline Structure flow stability factor (1.0-3.0):	1
Weir flow submergence decay exponent (1.0-3.0):	1
Gate flow submergence decay exponent (1.0-3.0):	1
DSS Messaging Level (1 to 10, Default = 4)	4
Maximum error in water surface solution (Abort Tolerance):	100

☐ Compute energy losses over junctions

OK Cancel Defaults ...

Any configuration parameters used in XPSWMM or changes to tolerances in XPSWMM and HEC-RAS should be documented and justified.

4.4 TWO-DIMENSIONAL MODELS

Often the hydraulics of a study reach is complex, where flow mixing is occurring and there are lateral flow connections. In such cases, a two-dimensional (2D) model may be considered. According to FEMA's *Guidance for Flood Risk Analysis and Mapping – Hydraulics: Two-Dimensional Analysis*, “2D models are most applicable when terrain is flat, floodplains are broad, water moves in two or more directions, and/or where flow is hydraulically disconnected between the main channel and the floodplain.”

2D dynamic modeling is intended for use when one-dimensional (1D) modeling is inappropriate for the study reach. A modeling approach is inappropriate when the theoretical assumptions of the 1D approach (e.g., flow reversals are insignificant, or flow is gradually varied) are violated by the flow conditions of the project. Engineering studies intending to use 2D models must document why the traditional 1D approach cannot be used and why a 2D model is required to comply with the scope of the study.

Multi-dimensional model software is typically classified using two of four terms that describe the mathematical solver methodology. A model is described as being either implicit or explicit and as being either finite element or finite difference. Each of the preceding classifications has data requirements and solution behaviors that present unique challenges; therefore, all stages of 2D model development, including the type of mathematical approach used by the model and the reasoning behind the model's selection for a study must be fully documented.

Two-dimensional models solve the shallow water equations describing flood wave propagation in two directions with the third dimension (vertical) being depth averaged. The shallow water equations (SWE), also known as the St. Venant equations or the dynamic wave equations, include the momentum equation and the continuity equation. 2D models may simplify the SWE depending on the equation set used (e.g., Diffusion Wave or Full Momentum). 2D models being submitted for FEMA map changes must adhere to the requirements in *Guidance for Flood Risk Analysis and Mapping – Hydraulics: Two-Dimensional Analysis*.

4.4.1 Domain Extents and Orientation

The domain extents of a 2D model must cover the entire project area and extend beyond the project far enough area to allow a solution for the floodplain without piling up water at the boundary or encountering any other boundary effect. The domain must also extend upstream and downstream far enough to tie in with water surface elevations of effective water surface models outside the project area within half a foot.

Grid orientation options can be used to align the computation grid with the project stream network. Orienting the grid provides the user with an opportunity to fine tune or optimize the model solver. If this option is used, however, the orientation of the X-axis should be kept between 90° and -90° of east.

4.4.2 2D Terrain

Model cell elevations can be derived from many sources. Whether a model can use a particular source format varies among software developers. The results from a 2D model are very sensitive to the quality of the underlying terrain model. Extra care should be used to ensure that the model terrain is the highest quality available for the scale of the study. Please note that terrain modifications tools within RASMapper are acceptable for approximate grading, but design level grading should be completed outside of HEC-RAS.

If the model cell elevations are determined from point elevation data, the cell elevations should represent an average of the points within a given cell.

If the cell elevations are determined from a grid or TIN, the individual cell elevations should represent an average of the underlying TIN or grid elevations.

The resulting computational domain should be reviewed carefully to ensure that elevation transitions smoothly throughout the domain. Abrupt changes in elevation will cause model instability.

4.4.3 Roughness

Experience is crucial in selecting roughness values for 2D flows. Published guidance is not readily available outside software reference manuals. Because 2D models are recommended for areas with complex flow patterns, they tend to include more shallow flow areas where flow is transferred between channels. Roughness values should be high when depth of flow is shallow and less likely to layover vegetation. This is why some 2D modelers have found that roughness values tend to be higher than those used for 1D models. This is especially true for rain on grid models where the 2D mesh is modeling early stages of runoff collection and channelization. The project engineer should use observed flows and/or high-water marks whenever available to calibrate the roughness values. If observed data is not available a sensitivity analysis, in conjunction with software documentation, should be used to determine reasonable roughness values.

4.4.4 Non-conveyance Areas

Non-conveyance areas should be set to reflect natural conditions as closely as possible. Techniques such as turning off cells, using high roughness or elevation values, and using thin walls to simulate non-conveyance areas for project design purposes is acceptable. However, removing elements from computation should not be used when developing Base Flood Elevations (BFEs) for mapping, per FEMA's *Guidance for Flood Risk Analysis and Mapping – Hydraulics: Two-Dimensional Analysis*.

4.4.5 2D Hydraulic Structures

Most 2D software products include bridge, weir, or culvert routines. However, bridges and culverts should be modeled externally to verify that the 2D results represent the hydraulic structure performance correctly and the differences should be documented accordingly. Refer to software reference manuals for specific guidance. Typically, 2D software model structures as a rating curve or as a 1D/2D interface.

4.4.5.1 Rating Curve Method

If the 2D model software being used requires a rating curve to represent the performance of a hydraulic structure, the project engineer should select an appropriate calculator and document the derivation technique used to create the rating curve. The study must also include an explanation of how the hydraulic structure is tied to the vertical datum, how the structure is aligned relative to the stream, and a list of the cells associated with the structure.

4.4.5.2 1D/2D Interface Method

Modeling software that relies on 1D/2D hybrid methods to represent hydraulic structures often uses abstract representations rather than physical geometry parameters for hydraulic structures. When these methods are used, the study should document how the model represents hydraulic structures in the study area. The performance of the hydraulic structures in 1D/2D models should also be compared to external software solutions for verification.

4.4.6 Boundary Conditions

2D models are dynamic models that minimally require upstream and downstream boundary conditions. The downstream boundary condition should consist of an energy grade slope, stage-time hydrograph, or rating curve. The upstream boundary condition should consist of a flow-time hydrograph. The modeler should also be aware that additional boundary conditions are required anywhere water enters or leaves the domain. No boundary condition should be used to restrict water from entering or leaving the domain without justification. All boundary conditions should be fully documented.

A stage hydrograph boundary condition can be used instead of normal depth if the defined hydrograph boundary can be defined from an unsteady simulation for the adjacent channel. A rating curve boundary condition can be used instead of normal depth if the defined boundary aligns to a modeled dam/pond where a water surface elevation is calculated in the hydrologic model.

Each boundary condition should be named so that it is reflective of the location within the model and the source of the data (i.e., inflow hydrographs should reflect the name of the hydrologic element used to determine the hydrograph).

4.4.7 Hydrologic Input

If the 2D model is being used to support a FEMA map change, the study must maintain all flow change location peak flows used in the effective model unless the study is changing the effective hydrology. Hard coding in hydrographs for boundary conditions should be avoided when possible as it can introduce error. Hydrographs should be linked between hydrologic and hydraulic software (e.g., DSS files in HEC-RAS and HEC-HMS).

4.4.7.1 Lumped Flow Routing (External Rainfall-Runoff)

Lumped flow routing models, like HEC-HMS, produce a hydrograph at junction points along a stream centerline. Routing losses are included at each junction; therefore, 2D models using lumped flow model output should be built in a manner to not include additional routing losses between junction locations if the model area encompasses more than one output node; to do so would double-count routing losses and produce erroneous results.

Hydrographs from external sources should be assigned across several domain elements, rather than assigning the entire inflow hydrograph to a single domain element, to minimize surging.

4.4.7.2 Distributed Flow Routing (Internal Rainfall-Runoff)

Some 2D software applications provide unit hyetograph and frequency storm routines that calculate watershed runoff and apply the resulting hydrographs to 2D domain elements or to nodes along a 1D/2D reach. The use of this method will be considered a change to the effective hydrology for regulatory purposes in the SARB. Proposals to use this method must be approved by the local community Floodplain Administrator, include areal reduction if the contributing area is greater than 10 square miles, and fully document why the effective hydrology method was inappropriate.

4.4.7.3 2D Rainfall (Rain on Grid)

2D rainfall applies a hyetograph to a polygon that covers all, or a portion of, the model computational domain to produce runoff. The use of 2D rainfall must be coordinated with the local community Floodplain Administrator. Studies proposing 2D rainfall must present justification to obtain approval. If 2D rainfall is approved, the project engineer must incorporate areal reduction in the model if the drainage area at the outlet is greater than 10 square miles. The use of 2D rainfall will be considered a change to the effective hydrology for regulatory purposes in the SARB. A discussion as to why the effective hydrology method was inappropriate must be included in the study.

4.4.8 Initial Conditions

Two common methods to establish initial conditions have been encountered in 2D software packages. The first method enables the user to manually set initial conditions throughout the system. The second method requires a warm-up run to establish initial water surface elevations and other initial conditions. All initial conditions and the methods used to derive them shall be thoroughly documented in the study.

4.4.8.1 Losses and Antecedent Soil Moisture

Hyetographs entered for 2D Rainfall can be total rainfall or excess rainfall depending on how losses are considered. Some software allows for spatially varied infiltration (based on soils and landuse) in the mesh and therefor total rainfall can be applied to the mesh. Some software does not include infiltration within the mesh and excess rainfall should be applied. Modelers must verify they are not double counting flow losses, and the methodology must be fully documented.

The set of submitted model runs should include a sufficiently long warm-up run that establishes starting soil moisture conditions for the mapping run unless the software provides for the setting of initial soil moisture by other means. The target level for soil moisture and the rationale for the selected soil moisture level must also be documented.

4.4.9 Cell Size and Time Step

Cell size should be set appropriately for the project terrain. Cells should be large enough to accurately represent the floodplain being studied and ensure a smooth transition in terrain elevation. Consideration of the balance between the required detail needed to describe the flood plain terrain and model

computational efficiency is important. Cell size should not be set to remove specific structures or lots from the floodplain. Channels should have at least 3 to 4 cells across major flow paths.

Cell size and time step are closely interrelated in 2D modeling. The Courant Number, C , is a good check to determine the adequacy of these two parameters in relation to one another. One form of the relationship is shown below:

$$C = \frac{\Delta t}{\Delta x}(\sqrt{2gH})$$

Where:

Δt = time step

Δx = grid size

g = gravitational acceleration

H = water depth

The software has the ability to perform 2D unsteady flow routing using either of the equation sets Shallow Water Equations (SWE) or Diffusion Wave Equations (DWE), with each option having its own corresponding Courant Number to meet.

Models using the SWE method, which is preferred for explicit solvers, should have the cell size and time step set so that the Courant Number is less than or equal to 1. Models using the DWE method, which is preferred for implicit solvers, should have the cell size and time step set so that the Courant Number is less than or equal to 2. This is user selectable, giving modelers more flexibility.

In general, the DWE equation allows the software to run faster and have greater flexibility properties, while SWE is applicable to a wider range of problems. Because users can easily switch between equation sets, each can be tried for any given problem to see which option is more suitable for the needed solution. The choice of equation set may change solution methods so always refer to latest documentation for software used to determine if it necessary to use the DWE or SWE as well as guidance on the proper Courant Number requirements.

4.4.10 Simulation Options and Tolerances

There are many options to set operational tolerances and other parameters for any 2D model and they are all set at default values. The project engineer should review the model defaults to ensure they are reasonable. Refer to the reference manual for the particular software being used simulation option and tolerance guidance. The threshold parameter for determining whether a model cell is wet or dry should be set at 0.01 foot in the SARB. Other thresholds can be used if approved by the community Floodplain Administrator. The rationale behind using other thresholds should be fully documented in the study report.

4.4.11 Stability and Continuity Criteria

Instability in a dynamic model appears in the result hydrographs as rapid variations between high and low values with time or spikes of excessively high or low values. Sometimes it is referred to as “bouncing”, “searching” or “spiking.” It is the result of the model failing to converge properly and should be reduced or eliminated whenever possible. A model that runs perfectly without any instability produces smooth, continuous hydrographs. Instability in the fringes of a hydrograph at the beginning or end of a run may be acceptable if the study can establish that the instability does not impact the overall accuracy of the model; however, instability at the hydrograph peak should always be resolved.

Continuity, on the other hand, is a measure of how well the model accounts for the inflow volume versus the outflow volume. 2D model continuity should be within 1 percent, while 1D/2D model continuity should be within 5 percent.

For 2D XPSWMM (XP2D) models, the user should start with a computation time interval of 5 seconds and reduce the time interval as necessary to achieve an acceptable continuity error of less than 2 percent. Model stability, based on this continuity error threshold, should be achieved using the XPSWMM Classic Engine (rather than the Extreme Engine). XPSWMM Extreme Engine computations utilize the TUFLOW Heavily Parallelized Compute (HPC) method which relies on adaptive time steps to create an “unconditionally stable” model, as described in the TUFLOW documentation (TUFLOW, 2024). As a result, the Extreme Engine may overlook continuity errors since the HPC computations force model stability.

Once model stability is achieved within the acceptable continuity error threshold using the XPSWMM Classic Engine, the XPSWMM Extreme Engine may be used to improve model run efficiency. The Extreme Engine enables multiple CPU and GPU processing and provides significantly quicker model run times compared to the Classic Engine. Any model results produced by the Extreme Engine should be compared to the Classic Engine model results to verify that the modeling results have not changed significantly. If the Extreme Engine model runs show significant changes compared to the Classic Engine results, the model should continue to use the Classic Engine.

4.5 COMBINING 1D AND 2D MODELS

1D/2D hybrid models do well in areas where stream channels are well defined and the channel flow satisfies 1D modeling assumptions, but flow in the overbanks has significant lateral movement that no longer satisfies 1D modeling assumptions.

Cross-section placement will generally follow 1D practice guidelines. The difference between 1D and 1D/2D models lies in cross-section length. Cross sections for 1D/2D models typically only represent the channel from left bank station to right bank station. However, cross sections may be extended into part of the overbanks where 1D modeling assumptions remain valid and the transition to 2D would occur at the point where 1D assumptions are no longer satisfied. In either case, the project engineer must remain cognizant of where the transition from 1D to 2D occurs to ensure the model solution represents natural flow conditions.

4.6 FLOODWAYS

If floodways are to be calculated for an area, the standard methodologies outlined in the HEC-RAS Hydraulic Reference Manual (USACE HEC, 2002) and FEMA’s *Guidance for Flood Risk Analysis and Mapping – Floodway Analysis and Mapping* publication should be followed. More recent versions of these publications may be used as they become available. Floodways should be determined using steady state, one-dimensional hydraulic models.

4.6.1 Floodways with Lateral Structures

For communities which define separate floodways not concurrent with the regulatory 1-percent floodplain, special consideration should be given to handling lateral structures around floodways. If a lateral structure removes flow from the main channel, the engineer should determine whether a floodway should be defined for the main channel only or for both the main channel as well as the receiving channel following the process shown in Figure 4-4.

If a separate hydraulic model for the receiving channel is available (with its own flows independent of the spill flows from the lateral structure), a floodway must be defined on both streams. If the hydraulic model for the receiving end of the lateral structure only receives spill flow from the lateral structure, then a floodway does not need to be defined.

If the full 1-percent AC storm event flow is kept in the mainstem model (no flow leaving through the lateral structure), and results in a WSE less the 1 ft greater than the optimized 1-percent AC storm event WSE (with flow leaving through lateral structure), then the floodway does not need to be defined on the receiving stream.

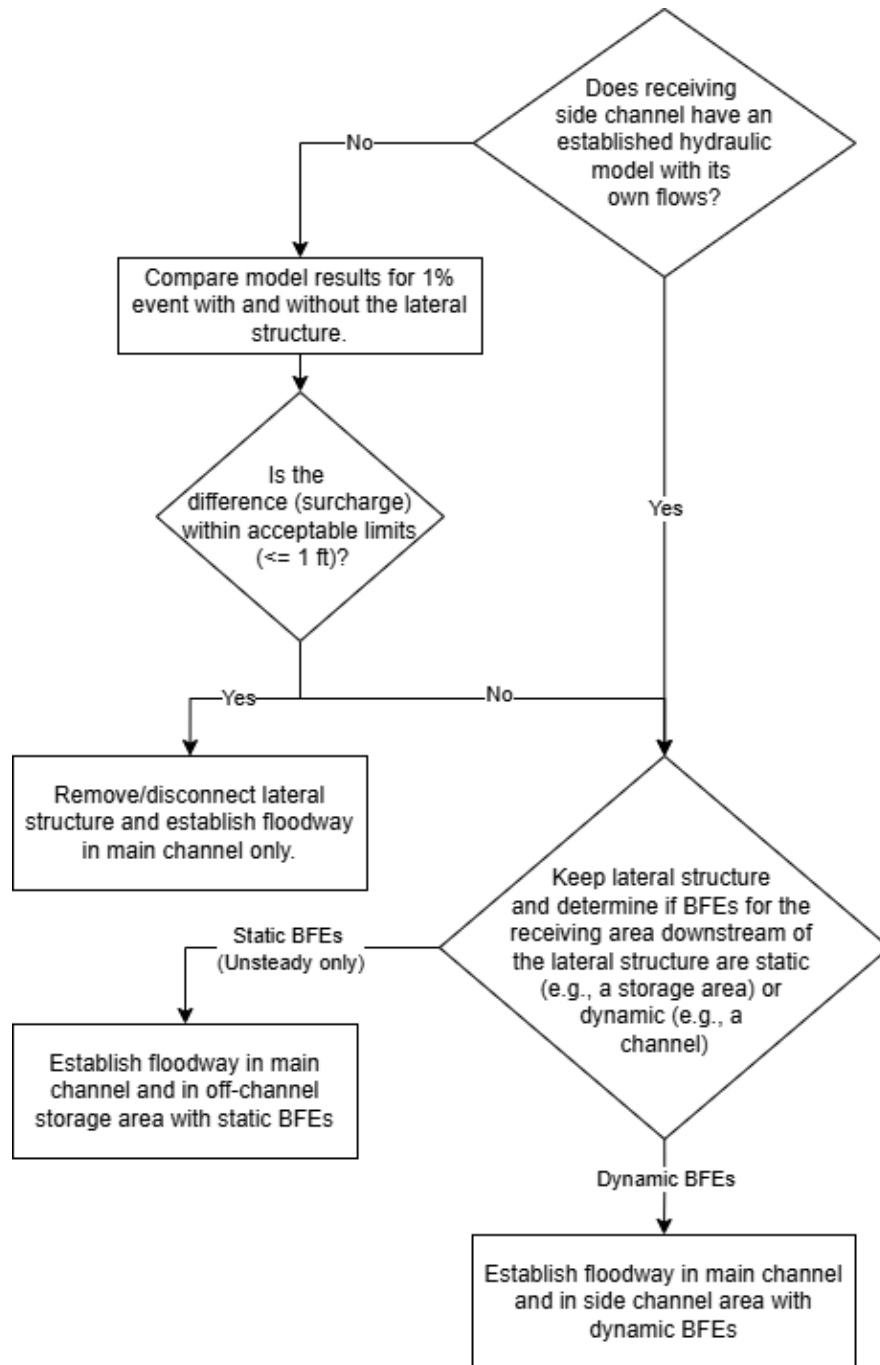


Figure 4-6 – Decision Flowchart for Floodway Development in Model with Lateral Structure

4.7 DOCUMENTATION

4.7.1 Model Metadata

In addition to the basic modeling standards described in the preceding sections, each model developed for the SARB must be thoroughly documented. This documentation should be in a form similar to the metadata required for geospatial data. It will include documentation of the source of the model, the spatial

extents, the methodologies used in the modeling, and any significant differences from previous models in the area. A summary of the additional information to be included in the modeling metadata is listed below.

- 1) Common Information
 - a) Responsible Engineer
 - b) Major Watershed
 - c) Brief Description (<250 characters)
 - d) Description of any unusual features
 - e) Purpose of study
 - f) Extents of modeling
 - g) General differences from preceding models
 - h) Date completed by engineer
 - i) Date of jurisdictional review
 - j) Date of River Authority review
 - k) File Name
- 2) Hydraulic Specific Information
 - a) Steady or unsteady flow simulation
 - b) Flow regime for model
 - c) Source of flows
 - d) Description of boundary conditions
 - e) Source of topographic data
 - f) Coordinate system and datum for topographic data
 - g) Source of roughness data

These items should serve as the minimum requirements for documentation of the models in the SARB. This information will be stored in a linked database and maintained by the River Authority so that it can be accessed to determine the source of data within the modeling geodatabase. This is in addition to the documentation of the modeling process required by FEMA for update of FIRM maps.

4.7.2 Geospatial Documentation

If the following elements are created or modified:

- Cross sections,
- Stream centerlines, or
- Structures

The supporting geospatial files are requested by the River Authority in the format outlined in Appendix E. The data should only be submitted for the elements being modified within the study area. Also see Section 2.1.3 for guidance on including terrain surface data with hydraulic models.

5 CALIBRATION

Calibration is vital for the development of accurate hydrologic and hydraulic models. All models accepted for use in the SARB should either be calibrated to historic data or validated to previously calibrated models. If using an uncalibrated model, the model should be calibrated based on historic data where available. Model calibration should be based on observed data, such as rainfall, flows, high water marks, or stage measurements. When using previously calibrated models, model validation may be performed by comparing the updated model to historic data, previously calibrated model results, or using alternate analysis methods (such as comparing results with regression calculations or other hydrologic methods).

Any proposed changes to *model methodologies*, as defined below in Section 5.1, must be coordinated with the River Authority and the Floodplain Administrator (FPA) to determine whether they will be an acceptable alternative to the existing methodologies. Acceptable model methodology changes should be verified through calibration or validation and justified in the associated model documentation. Where different models or methods meet along a reach, WSEs for all recurrence intervals shall tie within 0.5 ft, per FEMA *Guidance for Flood Risk Analysis and Mapping (General Hydraulic Considerations)*.

Variances to these requirements will be considered on a case-by-case basis with appropriate documentation and justification.

5.1 CHANGING MODEL METHODOLOGIES

The following section defines what constitutes a change in *model methodology* for hydrologic and hydraulic models. Any changes to model methodologies require coordination with the River Authority and FPA for acceptance of the proposed changes and to determine if additional model calibration is required. Model changes not listed in these sections will be considered on a case-by-case basis and should include appropriate documentation.

5.1.1 Changes to Hydrologic Model Methodologies

For a hydrologic model, the following types of model changes are considered changes in methodology requiring special coordination with the FPA and potential model calibration:

- Software (e.g., changing between HEC-HMS and XPSWMM)
- Transform/lag time (e.g., changing from Snyder Unit Hydrograph method to an alternate transform method)
- Infiltration (e.g., changing from Curve Number to an alternate infiltration method) (note that the loss method should *not* be updated if the model was previously developed using the Green and Ampt loss method; for additional guidance on modifying Green and Ampt parameters, refer to Section 3.4.2)
- Rainfall (e.g., using alternate rainfall data or methods, including any changes to areal reduction methods)

The following types of hydrologic model changes are *not* considered changes in methodology and generally do not require coordination with the FPA:

- Updating to latest software version
- Redelineating subbasin boundaries
- Updating subbasin parameters (e.g., transform/lag time or infiltration) using previously accepted methods
- Updating channel routing parameters
- Changing model time steps

For further guidance on optimization for hydrologic modeling, please refer to *USACE Hydrologic Modeling System HEC-HMS User's Manual* (USACE, 2018).

5.1.2 Changes to Hydraulic Model Methodologies

For a hydraulic model, the following types of model changes are considered changes in methodology requiring special coordination with the FPA and potential model calibration:

- Software (e.g., changing between HEC-RAS and XPSWMM)
- Changing model analysis type (e.g., changing between 1D, 2D, or 1D/2D hydraulic models, incorporating rain-on-grid methods, or changing between steady or unsteady hydraulic models)

The following types of hydraulic model changes are *not* considered changes in methodology and generally do not require coordination with the FPA:

- Updating to the latest software version
- Manning's n values
- Changes to boundary conditions
- Adjustments to flow change locations
- Model time step settings
- 2D model cell sizes
- Terrain

5.2 MODEL CALIBRATION

The following section describes acceptable methods for performing model calibration including specific guidance for hydrologic models (Section 5.2.1) and hydraulic models (Section 5.2.2).

Depending on measured parameters available, attempts should be made to calibrate hydrology before calibrating hydraulics. Peak flows should be calibrated as closely to measured data as possible. When possible, additional measured data should be collected to augment the available calibration data sets. Calibration results within the following tolerances are acceptable:

- Peak flows within 10-percent of observed flows,
- Timing of the peak within 30 minutes (or less for small watersheds), and
- Water surface elevations within 0.5 feet of measured stage information

For calibrating steady state models, flows and timing of the peak are typically verified in the hydrologic model, while water surface elevations are verified in the hydraulic model, *FEMA Guidance for Flood Risk Analysis and Mapping (Hydrology: Rainfall-Runoff Analysis)*. For calibrating unsteady state models (1D and 2D), flows, timing of the peak, and water surface elevations are typically all verified in the hydraulic model, *FEMA Guidance for Flood Risk Analysis and Mapping (Hydraulics: One-Dimensional Analysis)*.

Coordination with the River Authority is recommended to determine the appropriate calibration limits and request concurrence on the calibration approach, available historic data, and selected historical storm events. The calibration extents should represent the area of influence, which depends on the magnitude of modeling changes and the available data. This area may include the study reach, the entire stream, or multiple streams.

Calibration should be performed based on a minimum of three historical events, and verification should be performed based on a minimum of two historical events. The models should also be verified and the calibration adjusted after significant new flooding events occur. The calibration events should be

significant events (equivalent to the 50-percent AC or larger design storm) preferably with large events (10-percent AC or greater) when sufficient data are available, *FEMA Guidance for Flood Risk Analysis and Mapping (Hydrology: Rainfall-Runoff Analysis)*. A relatively recent set of floods events should be chosen to reflect recent land use changes. If the storm occurred more than five years in the past, the model should be adjusted to account for the subsequent changes in land use. This is especially important in the San Antonio area. More recent events will also allow for the use of radar rainfall data.

Hydraulic models can be used to estimate peak flows based on such stage or high-water mark information. A calibrated hydrologic model may be used to estimate the flow at locations for which flow is not directly measured. Calibration of a set of hydrologic and hydraulic models is a linked process and some iteration between the two may be required.

In addition to calibration based on measured historical events, the design storm flows should be checked for reasonableness. Flood frequency analyses at all available USGS gauges within the study area should be performed with up-to-date data. Flood frequency analysis needs to conform to latest version of USGS Bulletin 17C.

5.2.1 Hydrologic Model Calibration

Adequate hydrologic model calibration consists of two primary components. These are the matching of measured peak flows and the matching of the timing of the observed peaks. The initial estimates of model parameters (CN, percent of impervious cover, lag time, and routing coefficients) should be adjusted in order to minimize the differences between measured and simulated events. USACE EM 1110-2-1417 includes a general procedure for tuning parameters during calibration. The HEC-HMS Technical Reference Manual also provides guidance on calibration procedures. In general, the parameters that are based on measured physical data, such as the percent impervious cover and the soil types, are not preferred for calibration purposes, and should be minimally varied when used for calibration, then those that are based on calculated or assumed conditions (e.g., lag time and routing coefficients). Antecedent moisture conditions must also be considered in the adjustment of loss rate parameters during the calibration process.

Rainfall data from NEXRAD radar should be ground-truthed and then used to provide better representations of the areal distribution of measured precipitation. If radar rainfall data is not readily available for a particular event, precipitation data from rain gauges should be distributed over the modeled watershed based on either the Thiessen polygon or Isohyetal methods.

5.2.2 Hydraulic Model Calibration

The primary calibration parameter in a hydraulic model is the Manning's roughness coefficient. Most of the other elements of the model are based as closely as possible on the measured real-world geometry of the stream and any structures along that stream. The Manning's roughness coefficients listed in Table 4.1 serve as an initial estimate for the channel roughness and may be adjusted upward or downward by a maximum of 0.005. For overbank areas, Manning's roughness coefficients may be adjusted within the ranges shown in Table 4.1 during calibration. Calibrated Manning's n (and other loss parameters) shall remain within the published ranges unless field evidence justifies otherwise, in accordance with *FEMA Guidance for Flood Risk Analysis and Mapping (General Hydraulics and Considerations)*. During calibration, the presence and behavior of ineffective areas, blocked obstructions, lateral structures, and storage areas should be reviewed, and appropriate changes should be made to the model if and when necessary.

Rainfall and stage measurements from USGS, Edwards Aquifer Authority, or COSA flood-warning sites and surveyed high-water marks should be used to guide the calibration of the hydraulic model. It is important to remember that most stage data will be available near existing structures and that these parts of a stream system are most susceptible to debris blockage. If modeled results are significantly different from the observed data at a structure, debris blockage is a possible cause that should be investigated to the highest extent possible. High-water marks are often surveyed by the USGS or other entities after significant flooding events. These may provide data in areas other than near structures.

6 FLOODPLAIN MAPPING

6.1 TOPOGRAPHIC WORKMAP

FEMA LOMC submittals require specific information to be included in a topographic workmap. Topographic workmaps submitted for flood studies within the River Authority jurisdiction intended for submittal to FEMA is required to include the following information:

- Boundaries of the effective floodplains and floodway as shown in the effective FIRM
- Boundaries of other modeled floodplains which are part of the study (ex: existing, proposed, ultimate)
- Location and alignment of all stationing control indicated
- Site boundary
- Parcel boundaries
- labeling of proposed hydraulic structure geometry
- cross sections and river stationing
- source(s) and effective date(s) of effective floodplains displayed on map
- Show and label streams roads and significant structures
- Community easements and boundaries
- Certification of a registered professional engineer
- Topographic contour information with labels
- Reference of the vertical datum
- Aerial background imagery

Topographic workmaps submitted for flood studies within the River Authority jurisdiction which are not intended for submittal to FEMA should include all information in the first bulleted list above along with the following information:

- Location of flow change locations or other locations of input flows
- Location of affected structures
- Labeling of existing hydraulic structure geometry (ex: culvert sizes, bridge locations, pier sizes, spillway weirs, etc.)

6.2 FLOODPLAIN

Floodplain mapping and the development of associated documentation will be based on FEMA standards. These standards are outlined in the FEMA's Guidelines and Standards for Flood Risk Analysis and Mapping. These include, but are not limited to the existing conditions 1-percent AC and 0.2-percent AC.

FEMA allows for the display of the future (ultimate) condition, 1-percent AC floodplain on FIRM panels. The guidelines described in the FEMA publication *Modernizing FEMA's Flood Hazard Mapping Program: Recommendations for Using Future Conditions Hydrology for the National Flood Insurance Program* (FEMA, 2001) and formalized in recent changes to the Code of Federal Regulation (44 CFR 59 and 64) may be used for the display of future condition floodplains on FIRM panels for the region.

6.3 GEOSPATIAL DOCUMENTATION

The following geospatial files are required in support of a Letter of Map Change:

- Floodplain (area and line formats)
- Base Flood Elevations or include 1-percent AC water surface elevation in cross section attributes
- Conditional Letter of Map Revision (CLOMR)/LOMR Boundary

The following geospatial files are also requested if the information was modified as part of the analysis:

- Contours (if produced for mapping included with the Letter of Map Change submittal)

Specific requirements for the geospatial documentation files are provided in Appendix E. The data should only be submitted for the elements being modified within the study area.

7 SEDIMENT TRANSPORT

Reserved

8 GLOSSARY

ARMORING: Formation of a resistant layer of relatively large particles resulting from removal of finer particles by erosion.

AVULSION: A sudden change in the course of a stream.

BAR: An elevated region of sand, gravel, or other material deposited by flow.

BED LOAD: Sediment transport mode in which individual particles either roll or slide along the bed as a shallow, mobile layer a few particle diameters deep, the part of the load that is not continuously in suspension.

CHANNEL MORPHOLOGY: The form of a channel (cross-section, profile, and plan) and its change with time.

HEADCUT: An erosional feature in a stream where an abrupt vertical drop, also known as a knickpoint in the streambed occurs.

9 WORKS CITED

- Arcement, G., & Schneider, V. (1989). *Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains*. Water Supply Paper 2339, USGS.
- Asquith, W., & Roussel, M. (2004). *Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas*. Scientific Investigations Report 2004-5041, USGS and TXDOT, Austin.
- Asquith, W., & Slade, R. (1996). *Regional Equations for Estimate of Peak-Streamflow Frequency for Natural Basins in Texas*. Austin, TX: USGS.
- Bouwer, H. (1969). Infiltration of Water into Nonuniform Soil. *Journal of the Irrigation and Drainage Division*. doi:<https://doi.org/10.1061/JRCEA4.0000669>
- Chang, H. (1998). *FLUVIAL-12*. Retrieved from San Diego State University, Software website: <http://www.advancedengineeringsoftware.com>
- Davidian, J. (1984). *Computation of Water-Surface in Open Channels*. USGS.
- FEMA. (2001). *Modernizing FEMA's Flood Hazard Mapping Program: Recommendations for Using Future-Conditions Hydrology for the National Flood Insurance Program*. Federal Insurance and Mitigation Administration, Hazards Study Branch. Retrieved from <http://www.fema.gov/library/viewRecord.do?id=2218>
- FEMA. (2003). Appendix A: Guidance for Aerial Mapping and Surveying. In *Guidelines and Specifications for Flood Hazard Mapping Partners*.
- FEMA. (2003a). Guidelines and Specifications for Flood Hazard Mapping Partners. Retrieved from <http://www.fema.gov/library/viewRecord.do?id=2206>
- FEMA. (2003b). Appendix A: Guidance for Aerial Mapping and Surveying. *Guidelines and Specifications for Flood Hazard Mapping Partners*. Retrieved from <http://www.fema.gov/library/viewRecord.do?id=2206>
- FEMA. (2009). *Appendix C: Guidance for Riverine Flooding Analysis and Mapping*. Retrieved from <http://www.fema.gov/library/viewRecord.do?id=2206>
- FEMA. (2016). *Guidance for Flood Risk Analysis and Mapping, Hydraulics: One-Dimensional*. Retrieved from https://www.fema.gov/sites/default/files/2020-02/Hydraulics_OneDimensionalAnalyses_Nov_2016.pdf
- FEMA. (2022). *Guidance for Flood Risk Analysis and Mapping, General Hydraulics Considerations*. Retrieved from https://www.fema.gov/sites/default/files/documents/fema_general-hydraulics-guidance_112022.pdf
- FEMA. (2024). *Guidance for Flood Risk Analysis and Mapping, Hydrology: Rainfall-Runoff Analysis*. Retrieved from https://www.fema.gov/sites/default/files/documents/fema_guidance_hydrology_rainfall-runoff_analysis_nov_2024.pdf
- FEMA. (2024). *Guidance for Flood Risk Analysis and Mapping, DataCapture - Workflow Details*. Retrieved from HYPERLINK "https://www.fema.gov/sites/default/files/documents/fema_guidance_data_capture_workflow_details_nov_2024.pdf" https://www.fema.gov/sites/default/files/documents/fema_guidance_data_capture_workflow_details_nov_2024.pdf
- FHWA. (2000). *BRISTARS*. Retrieved from Federal Highway Administration, Transportation Research Board, Software website: <http://trid.trb.org/view.aspx?id=654095>
- FHWA. (2001, March). *Stream Stability at Highway Structures*. National Highway Institute.
- Flynn, K., Kirby, W., & Hummel, P. (2006). *User's Manual for Program PeakFQ, Annual Flood-Frequency Analysis Using Bulletin 17B Guidelines*. Reston, VA: USGS.

- Frederick, R., Myers, V., & Auciello, E. (1977). *Five- to 60-Minute Precipitation Frequency for the Eastern and Central United States*. Technical Memorandum HYDRO-35, NOAA, National Weather Service, Silver Spring, MD.
- Hershfield. (1961). *Rainfall Frequency Atlas of the United States*. Technical Paper No. 40, USDA Soil Conservation Service, Washington, DC.
- NCHRP. (2004). *Handbook for predicting stream meander migration*. Washington, D.C.: Transportation Research Board.
- NOAA. (1982). *Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian*. Hydrometeorological Report No. 52, Washington, DC.
- NRCS. (2004). Chapter 9 Hydrologic Soil-Cover Complexes. In *National Engineering Handbook* (Vol. Part 630 Hydrology).
- NRCS. (2007a). Chapter 16 Hydrographs. In *National Engineering Handbook* (Vol. Part 630 Hydrology).
- NRCS. (2007b). Stream Restoration Design. In *National Engineering Handbook, Part 654*.
- NRCS. (2010). Chapter 15 Time of Concentration. In *National Engineering Handbook* (Vol. Part 630 Hydrology).
- Ockerman, D. J. (2002). *Simulation of runoff and recharge and estimation of constituent loads in runoff, Edwards Aquifer recharge zone (outcrop) and catchment area, Bexar County, Texas, 1997–2000*. U.S. Geological Survey Water-Resources Investigations Report 02–4241.
- Ockerman, D. J. (2007). *Simulation of streamflow and estimation of ground-water recharge in the upper Cibolo Creek watershed, south-central Texas, 1992–2004*. U.S. Geological Survey Scientific Investigations Report 2007–5202.
- Pemberton E. L. and Lara, J. M. (1984, January). Computing Degradation and Local Scour. *Technical Guideline for Bureau of Reclamation*, 48.
- Perica, S., Pavlovic, S., St. Laurent, M., Trypaluk, C., Unruh, D., & Wilhite, O. (2018). *NOAA Atlas 14, Volume 11, Version 2: Precipitation-Frequency Atlas of the United States, Texas*. Silver Spring, MD: NOAA, National Weather Service.
- PSIAC. (1968, October). *Factors Affecting Sediment Yield and Measures for the Reduction of Erosion and Sediment Yield*. Pacific Southwest Inter-Agency Committee.
- SARA. (2013, May). *Natural Channel Design Protocol, San Antonio Texas Region*. San Antonio River Authority.
- SARA. (2019, December). *SARA Land Use Project Methodology*. Retrieved from GIS Open Data Website: <https://sariverauthority-sara-tx.opendata.arcgis.com/documents/1c25733c5ca44743acdf0b1833ed2e0b/about>
- Thomas, L. E. (1986). *Hydrology Technical Report No. 4 - Time of Concentration*. Chester, PA: NRCS.
- TUFLOW. (2024, January). *HPC Introduction*. Retrieved from TUFLOW Wiki: https://wiki.tuflow.com/HPC_Introduction
- TXDOT. (2001). *Hydraulic Design Manual*. Retrieved from <http://onlinemanuals.txdot.gov/txdotmanuals/hyd/index.htm>
- USACE. (1989, December). Sedimentation Investigation of Rivers and Reservoirs. *USACE EM No. 1110-2-4000*.
- USACE. (1993). *Appendix D River Modeling - Lessons Learned*. Engineering Manual 1110-2-1416.
- USACE. (1994). *Flood-Runoff Analysis*. Engineering Manual 1110-2-1417, Engineering and Design.
- USACE. (1999, July). *Channel Rehabilitation: Processes, Design, and Implementation*.

9 Works Cited

- USACE. (2002). *SAM Hydraulic Design Package for Channels, SAMwin software*. Vicksburg, Mississippi: Coastal and Hydraulics Laboratory-Engineer Research Development Center, Waterways Experiment Station.
- USACE HEC. (1996). *Runoff Simulation Using Radar Rainfall Data*. Technical Paper 155, Davis, CA.
- USACE HEC. (2000). *Hydrologic Modeling System HEC-HMS Technical Reference Manual*. Washington, DC.
- USACE HEC. (2002). *HEC-RAS, River Analysis System Hydraulic Reference Manual*. Davis, CA.
- USACE HEC. (2010, January). *HEC-RAS, River Analysis System User's Manual, Version 4.1*.
- USBR. (2006a, November). *Erosion and Sedimentation Manual*. US Bureau of Reclamation, Department of the Interior.
- USBR. (2006b). *Sediment Impact Analysis Methods (SIAM)*. Retrieved from US Bureau of Reclamation, Department of Interior, Software website: <http://www.usbr.gov/pmts/sediment/model/srhsiam/>
- USDA. (2012). *Soil and Water Assessment Tool*. Retrieved from United States Department of Agriculture, Agriculture Research Service: <http://swat.tamu.edu/software/>
- USDA. (2024). *Soil Texture Calculator (Multi-Point Texture Triangle)*. Retrieved December 2024, from NRCS Resources: <https://www.nrcs.usda.gov/resources/education-and-teaching-materials/soil-texture-calculator>
- USGS. (2018). *Guidelines for Determining Flood Flow Frequency - Bulletin 17C*. U.S. Geological Survey Techniques and Methods.
- UTSA. (2008, February). *Technical and Field Guide: Management Practices for Natural Waterways*. Center for Water Research, University of Texas at San Antonio.

Appendix A: Maps of Model Detail

**Reserved - For more information, see
Digital Data Modeling Repository (D2MR)**

Appendix B: Drainage Basin Delineation maps

**Reserved - For more information, see
Digital Data Modeling Repository (D2MR)**

Appendix C: Development of Design Rainfall Information



An employee-owned company

March 3, 2005

Mr. Nefi Garza, P.E.
San Antonio River Authority
100 East Guenther Street
San Antonio, Texas 78204

RE: **Project # 441184.02 and 441184.04**
General Hydrologic and Hydraulic Modeling Tasks: Development of Design
Rainfall Information

Dear Nefi:

The following is a brief report summarizing the design rainfall information to be used for the San Antonio River Basin. The report describes proposed guidance for the use of point rainfall versus an ellipsoidal distribution, a proposed methodology for the determination of point rainfall totals for various durations at any location in the River Basin, and proposed rainfall totals based on the new USGS report. The use of HEC-1 instead of HEC-HMS in order to facilitate areal reduction based simulations is also discussed.

Point Rainfall and Ellipsoidal Distribution

Point rainfall is recommended for use in the design storm simulations for all watersheds tributary to the Medina and Lower San Antonio Rivers. An ellipsoidal rainfall distribution should be used for the Medina River downstream of Medina Lake and the San Antonio River downstream of the confluence with the Medina River. It may also be advisable to use the ellipsoidal distribution for the lower portion of the Cibolo Creek watershed below Interstate Highway 10 East given the size and elongated shape of the watershed.

The use of point rainfall to define the design for the tributary watersheds of the major streams is reasonable given the size of these watersheds. The San Antonio River upstream of the confluence with the Medina River is one of the largest at 372 square miles (this area includes the 223 square mile Salado Creek watershed). This approach, when used with the areal reduction techniques in either HEC-1 or HEC-HMS, will likely produce more conservative flows than would be produced by the proposed ellipsoidal approach. The ellipsoidal approach provides a greater reduction in rainfall with increasing areas as opposed to the areal reduction curves used by the models (basically the curves from TP-40).

The added complication of the ellipsoidal approach is not warranted for use in modeling of the tributary systems. While the ellipsoidal approach has been made as simple as possible, it still requires the evaluation of two storm orientations centered at multiple points within the subject watershed. The critical centering is also likely to be different for points within the watershed. As a result, the ellipsoidal approach is more appropriate for large basin-scale modeling, preferably with more generalized models than those used for the tributary watersheds.

Determination of Point Rainfall within the San Antonio River Basin

Point rainfall totals for the relevant design events have been developed for the five counties that comprise a majority of the San Antonio River Basin. These five counties from upstream to downstream are Bandera, Bexar, Wilson, Karnes and Goliad. It is recommended that the rainfall developed for these five counties be used to define the design rainfall for all drainage basins (defined by hydrologic unit) predominantly within each county. Refer to Figure 1 for the hydrologic units associated with each county.

The proposed methodology is a compromise approach that allows major variations in rainfall characteristics to be represented across the Basin, while remaining consistent and simple to apply for the majority of smaller watershed studies. As shown on Figure 1, most of the area to be studied as part of the Bexar County FEMA restudy would use the rainfall totals for Bexar County under the proposed methodology. The Bandera rainfall totals would be used for modeling of the Medina River upstream of Medina Lake. The Bandera rainfalls should be used with the ellipsoidal distribution simulation of flows along the Medina downstream of the dam.

Development of Point Rainfall Totals from USGS Report

The rainfall totals developed for the five counties are based on the USGS report "Atlas of Depth-Duration Frequency of Precipitation Annual Maxima for Texas" (Scientific Investigations Report 2004-5041). Rainfall totals for the 10-, 25-, 50-, 100- and 500-year events for durations of 5, 15, and 30 minutes; 1, 2, 3, 6, 12 and 24 hours; and 2, 3, 5 and 7 day durations were read from the atlas maps provided in the USGS report. This rainfall information is intended to be used for both the Frequency Distribution hyetograph and for determination of the rainfall totals and hyetographs shapes to be used with ellipsoidal storms.

Three values were determined for each recurrence and duration combination for each of the five counties. The three values consisted of the minimum and maximum rainfall totals at any point within a county and a representative value for the county. The representative values were determined based on an interpretation of the average rainfall total across a particular county. These values were tabulated and then graphed for each recurrence interval event in each county. The resultant graphs provided an initial estimate of the IDF curves and minimum and maximum boundary ranges for each of the IDF curve ordinates.

The initial IDF curve values were evaluated and revised as necessary in order to provide smooth curves for use in hydrologic modeling. The representative values determined from the atlas maps were adjusted as appropriate in order to smooth the curves. In most cases, the adjusted values fell within the minimum and maximum bounding ranges determined for the county. In a few cases, the values had to be adjusted above or below the bounding range in order to produce a smooth curve. This approach was discussed with William Asquith, author of the USGS report. He agreed in principle to the approach as described above. The resulting rainfall tables (Tables 1-5) and graphs (Figures 2-6) are attached at the end of this report. The preliminary and final tables and graphs are included in the spreadsheet on the CD attached with this report.

The 5-minute rainfall values called for in the Frequency Distribution hyetograph method (both HEC-HMS and HEC-1) are not available in the USGS report (15-minute is the smallest reported duration). The 5-minute values listed in the final table were developed under the

assumption that the 5-minute rainfall total would be approximately 37% of the 30-minute total. This approach is based on the guidance provided in Table 3 in TP-40. The table states that the 5- and 15-minute rainfall can be estimated as 37% and 72% of the 30-minute rainfall respectively. Guidance for the determination of the 5-minute rainfall was also provided by Craig Lofton of the USACE. He stated that the Corps typically uses a value between 46 and 50 percent of the 15-minute rainfall total. Based on the TP-40 approach, the 5-minute rainfall value for the 100-year event in Bexar County is 1.13 inches. This equates to 45% of the 15-minute rainfall determined from the USGS report, which falls just outside of the range used by the Corps. The ratio is the same as that obtained in consideration of the UDC 5- and 15-minute rainfall totals.

Comparison to Other Data Sets

The rainfall data developed as described above was compared with both the existing rainfall totals in the CoSA UDC, the TxDOT IDF curve equations with coefficients for the subject counties (derived from TP-40), Hydro-35 and TP-40. The results of these comparisons for the Bexar County 100-year event are shown in Table 6. The comparison is shown graphically in Figure 7. Equations in the form used for the TxDOT IDF curves also were fit to the unadjusted USGS rainfall totals. Equations for Bexar County were fit to both the full set of rainfall totals through the 7-day duration and from the 5-minute through 24-hour durations.

Table 6: Summary of IDF Rainfall Values for the Bexar County 100-Year Event

Duration	100-Year Rainfall Totals					
	UDC	TxDOT	USGS Adjusted	USGS Fitted (All)	USGS Fitted (24-Hour)	TP-40
5 minute	0.87	1.21	1.13	0.94	1.00	1.24
15 minute	1.91	2.40	2.50	2.17	2.24	2.41
30 minute		3.30	3.05	3.29	3.33	3.35
1 hour	4.25	4.26	4.35	4.52	4.50	4.27
2 hour	5.57	5.28	5.80	5.74	5.67	5.28
3 hour	6.23	5.92	6.60	6.42	6.35	5.86
6 hour	7.13	7.11	7.50	7.56	7.51	7.10
12 hour	8.05	8.46	8.80	8.69	8.72	8.55
24 hour	9.91	10.02	10.00	9.88	10.01	9.90

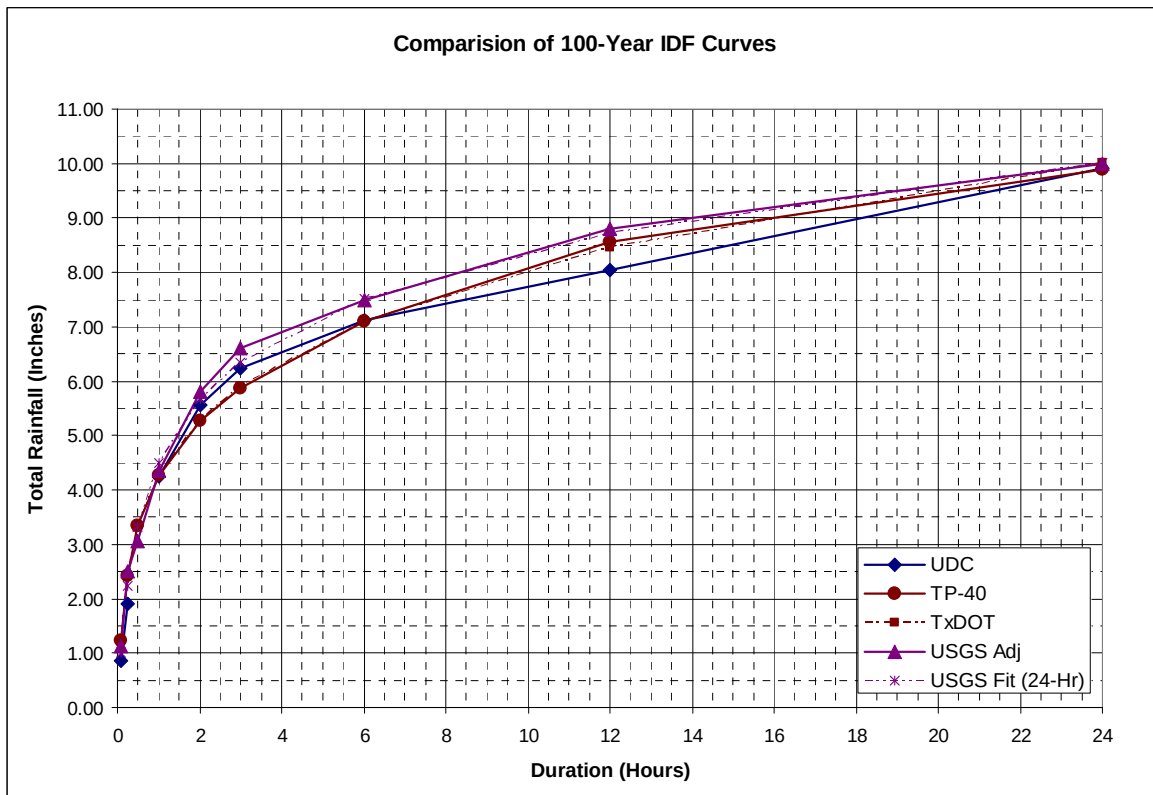


Figure 7: Comparison of IDF Curves for Bexar County

As can be seen in the table and figure, the most significant differences between the adjusted USGS values and the UDC values are in the 5- and 15-minute durations. The 1-hour through 24-hour values are also higher. However, the difference in the short duration events produce the largest difference in computed flows. The various IDF data sets were tested with a preliminary HEC-HMS model for the Salado Creek watershed. A HEC-HMS meteorologic model with the frequency storm distribution was developed for each version of the IDF curves. The results from the various meteorologic models produced total flows at the mouth that ranged from 2% to 6% higher with the USGS adjusted values producing the highest flows.

The differences were slightly more pronounced when looking at the individual drainage basins. All of the other IDF values produced substantially higher flows for drainage basins with short lag times. This is a result of the large relative differences between the UDC and other 5- and 15-minute rainfall values. In the case of the USGS fitted values, the difference in flows varies between 4 and 6 percent for drainage basins with lag times greater than 30 minutes. The differences tend to increase as the lag time decreases with a flow increase of approximately 30 percent for lag times around 15-minutes.

The 5- and 15-minute UDC values appear to be based directly on the Hydro-35 National Weather Service Publication. Both TP-40 and the USGS report indicate a significantly higher 15-minute rainfall than that derived from Hydro-35. This fact along with the 30-minute rainfall from the USGS report yield correspondingly higher 5-minute rainfalls. Even the equations fitted to the USGS values produce significantly higher 5- and 15-minute rainfalls.

Nefi Garza, P.E.

General H&H Modeling Tasks: Design Rainfall Information

March 3, 2005

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The USGS fitted values are recommended for use in hydrologic modeling for the Bexar County FEMA Restudy and for the Regional Watershed Modeling System. However, everyone should be aware that these values will, in general, produce higher flows than those produced by the UDC rainfall values.

Ellipsoidal Rainfall Distribution

The assumption of a single point rainfall begins to break down when considering larger and larger areas. Ellipsoidal storm distributions are recommended for determination of flows along major streams with large watershed areas (greater than approximately 500 square miles) or very elongated shapes (Cibolo Creek). For the San Antonio River Basin, such streams would be the Medina River below Medina Dam, The San Antonio River below the confluence with the Medina River and Cibolo Creek below IH10 East. Point rainfall values would still be used to determine the FEMA and design flows for all other tributaries. The ellipsoidal analysis would be focused on the development of FEMA and design flows along the main stems of the three mentioned streams.

The size and shape of the elliptical storms to be used were developed based on HMR-52 guidance. Two storm orientations were considered for the River Basin, one at 220 degrees and the other at 320 degrees (see Figures 8 and 9 below). These orientations were based on the guidance in HMR-52 and on analysis of isohyetal maps from recent, large rainfall events. These two orientations correspond to the major storm patterns typical of the area (large fronts moving from the southwest and tropical storms moving from the southeast). The values of the elliptical isohyets are derived from the maximum point rainfall for a given design storm based on the location of the center of the storm. If the storm is centered over the Bexar County areas as described above, the Bexar County value would be used as the rainfall total for the central 10-square mile ellipse. The rainfall values for the outer isohyets are then reduced according to areal reductions factor based on the area of the isohyets. The methodology used for the areal reduction was based on the USGS Water-Resources Investigations Report 99-4267, "Areal-Reduction Factors for the Precipitation of the 1-Day Design Storm in Texas."

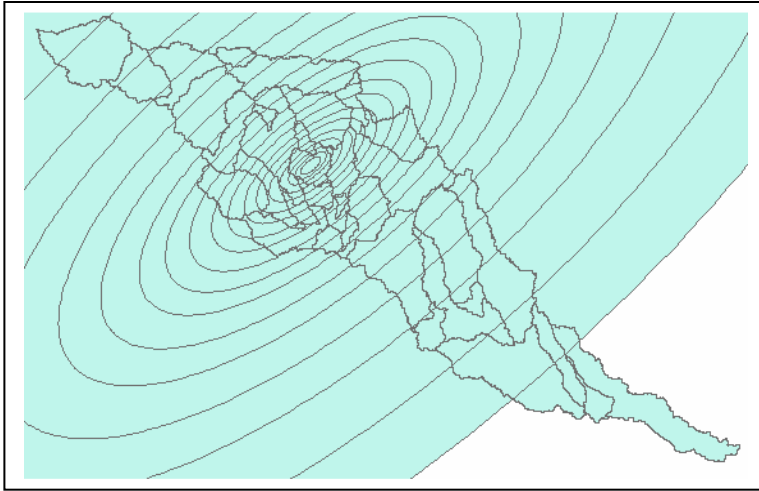


Figure 8: Ellipsoidal Orientation at 220 Degrees

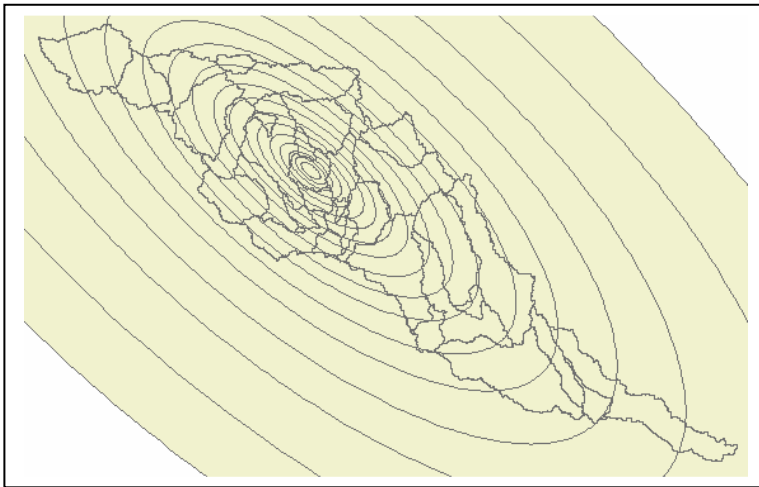


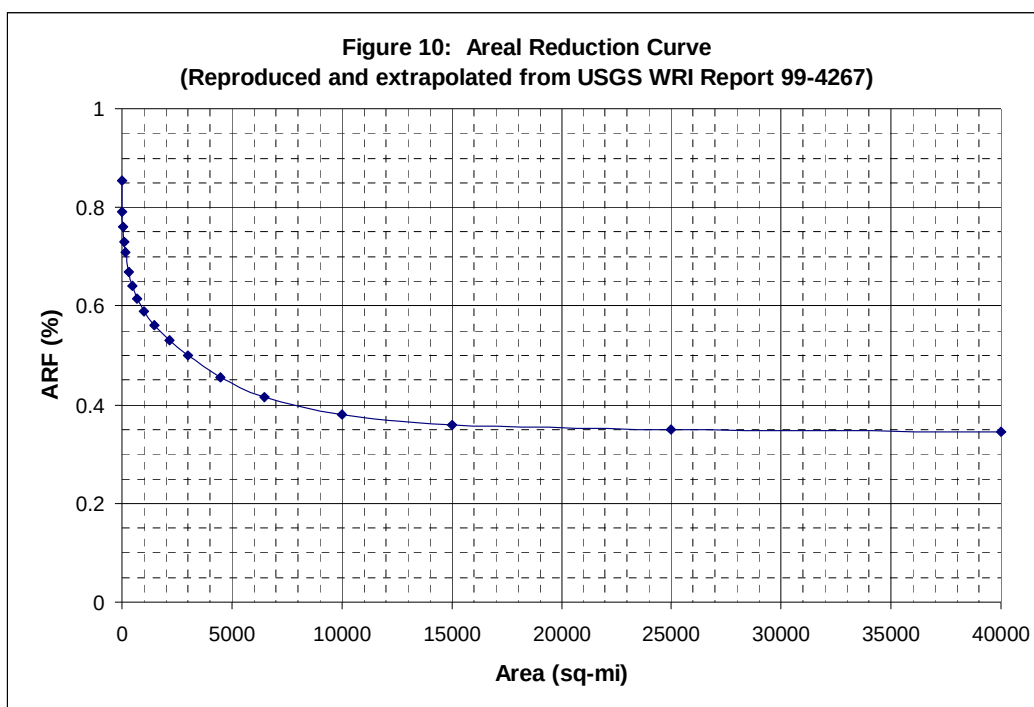
Figure 9: Ellipsoidal Orientation at 320 Degrees

Areal reduction is an important consideration in the use of both point rainfall and an ellipsoidal distribution. Recent work by Halff Associates on the Lower Colorado River Basin (Lower Colorado River FEMA Study) has shown that the areal reduction curves from TP-40 (equivalent to those used in HEC-1 and HEC-HMS), when applied to an ellipsoidal storm distribution, generate flows considerably higher than is reasonable based on statistical analysis of gage records. The maximum areal reduction in the 100-year event based on the TP-40 curves is about 90%. This would mean that for a rainfall centered on the upper Medina River, the rainfall at Goliad would still be 90% of the maximum rainfall. In order to address this overly conservative assumption, areal reduction curves from the USGS study are recommended for use in determining the rainfall values associated with each ellipse. These curves yield a maximum areal reduction to 34.5% of the centroidal point rainfall for the outermost ellipse (40,000 square miles). Refer to Table 7 and Figure 10 for a full summary of the areal reduction factors for the ellipsoidal storms.

Table 7: Areal Reduction Curve (Reproduced and extrapolated from USGS WRI Report 99-4267)

Ellipse	Area (sq mi)	Base ARF for Area	Reduction Factors for Ellipses
A	10	0.855	1.000
B	25	0.79	0.823
C	50	0.76	0.775
D	100	0.73	0.745
E	175	0.71	0.720
F	300	0.67	0.690
G	450	0.64	0.655
H	700	0.615	0.628
I	1000	0.59	0.603
J	1500	0.56	0.575
K	2150	0.53	0.545
L	3000	0.5	0.515
M	4500	0.455	0.478
N	6500	0.415	0.435
O	10000	0.38	0.398
P	15000	0.36	0.370
Q	25000	0.35	0.355
R	40000	0.345	0.348

Note: Curve extrapolated for ellipsoidal areas O, P, Q and R



In order to apply the ellipsoidal rainfall to a watershed, the ellipsoidal storm pattern must be used to calculate weighted rainfall for each drainage basin. The ellipsoidal storm patterns were initially created in CAD and then converted to shape files, one for the 220 degree orientation and one for the 320 degree orientation. GIS tools were then used to create appropriate attributes for the ellipsoid bands. The bands of the ellipsoids were attributed with the average value of the bounding isohyetal lines. In order to calculate the weighted rainfall for each drainage basin in a watershed, the ellipsoid shape file must be shifted to the appropriate centering point and then intersected with the shape file representing the drainage basins in the model. The weighted average storm rainfall for each of the watersheds may then be calculated for a given storm centering and orientation option.

The steps described above generate a weighted average rainfall total for each drainage basin in a model that must then be distributed over a hyetograph. A set of unit hyetographs was developed with the HMS model based on the frequency storm distribution and the Bexar County IDF values described in the preceding section. The ordinates of the resulting balanced hyetographs were divided by the total rainfall in order to obtain unit hyetographs. These unit hyetographs were developed for 24-hour, 2-day, 4-day and 7-day durations. The selection of the duration for use in a given analysis is dependent on the contributing watershed area at the point of interest. For the upper portions of the Medina and Cibolo, the 24-hour or 2-day durations may be adequate. For the San Antonio River between the confluence with the Medina and the confluence with the Cibolo, the 2- or 4-day duration would be more appropriate. Points below the confluence with Cibolo Creek should use either the 4- or 7-day duration. The duration used is dependant on the travel time of the peak from the upper portion of the water shed. In all cases, the simulation time should be approximately twice the rainfall duration in order to simulate the full runoff response of the watershed.

The weighted, total rainfall values calculated from the intersection of the basin watersheds with the storm centered ellipsoid, are distributed over these unit hyetographs to provide the necessary areally reduced precipitation for the model. This results in a separate hyetograph for each drainage basin that must then be assigned to a gage within HMS. Given the potential number of drainage basins and the number of centerings that must be analyzed, this process would be very tedious if done by hand. In order to automate this task, an application was created to distribute the calculated average precipitation values (manipulated in a spreadsheet and saved to a comma separated variable file) over the appropriate unit hyetograph. The resulting hyetographs for each drainage basin in the model are written out to a text file that is formatted for batch-mode entry into DSS. An external DSS rainfall input file containing the incremental hyetograph for each of the drainage basins may then be created for each of the storm centering options using the USACE DSSTS executable DOS program with the batch input files created by the application.

A gage must then be created in HMS for each drainage basin. Each of these gages must point to the directory containing the external DSS rainfall input files for the storm centering option. The various storm centering options may then be evaluated by simply performing a find and replace routine in the HMS gage file to replace the previous storm centering DSS file with the current storm centering DSS file of interest. The HMS model may then be executed and the global summary table with peak flows copied and pasted into an excel file for evaluation.

Nefi Garza, P.E.

General H&H Modeling Tasks: Design Rainfall Information

March 3, 2005

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Potential Use of HEC-1 Instead of HEC-HMS

The use of the HEC-1 model in place of HEC-HMS has been discussed in previous meetings. There are two primary advantages that have been put forth for this approach. The first is that detailed HEC-1 models already exist for portions of the Bexar County study area. Some of these models are quite complex and could be difficult to translate to HEC-HMS. The second is that HEC-1 offers the capability for consistent areal reduction whereas HEC-HMS will require significantly more runs or additional post processing in order to generate appropriate, areally reduced flows at all points within a watershed. The second factor is the more persuasive argument. However, models that are created or remain in HEC-1 will not be directly compatible with the RWMS. These models will need to be converted at some point in order to be integrated into the system. Ideally, version 3 of HEC-HMS will be soon become available and will eliminate the problems of consistent areal reduction in HMS. It is our recommendation that all hydrologic models, with the possible exception of the complex upper San Antonio River watershed, be created in HEC-HMS. This will facilitate integration into the RWMS and will also make the models easier to update in future studies.

If you have any questions or comments, please do not hesitate to contact me at (512) 342-3295.

Sincerely,

Karl McArthur, P.E.

Senior Water Resources Engineer

Attachments

cc: Tony Trollope, Duke Altman, Saul Nuccitelli, PBS&J
William Burmeister, Steve Gonzales, SARA

Appendix D: Development of an Alternative Storm Hyetograph Method for Areal Reduction

1965-2005



BEXAR COUNTY MAP MODERNIZATION

Development of an Alternative Storm Hyetograph Method for Areal Reduction

SEPTEMBER 2005

PAPE-DAWSON ENGINEERS, INC.

555 East Ramsey | San Antonio, Texas 78216 | Phone: 210.375.9000 | Fax: 210.375.9010 | info@pape-dawson.com

BEXAR COUNTY MAP MODERNIZATION

ARF STORM HYETOGRAPH METHOD

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7. Comparison of Current Model with Alternative 2 Model

Written By

Wayne Tschirhart, Troy Dorman, Shiva Sandrana, Robin Buske

BEXAR COUNTY MAP MODERNIZATION

ARF STORM HYETOGRAPH METHOD

Overview

As part of the Regional Watershed Modeling System (RWMS), new adjusted rainfall developed by PBS&J and based on a recent USGS study for the Texas Department of Transportation was adopted for use in hydrologic models. PBS&J also proposed an ellipsoidal rainfall distribution for areal reduction to be applied to specific watersheds in the San Antonio River Basin. Development of both the adjusted rainfall and ellipsoidal distribution are documented in the RWMS technical memorandum titled “Snyder Hydrograph Guidelines”, hereafter referred to as PBS&J-2005. The updated rainfall and ellipsoid areal reduction was tested using the San Antonio River (SAR) LMMP model. Analysis of the model output showed a 20% increase in flow using the new rainfall values with no other model changes. The increase was the result of an up to 30% increase in rainfall at short durations (<15min). Application of the ellipsoidal distribution reduced flows in the SAR basin to reasonable values but created a discontinuity at the junction of the San Antonio River and Medina River. A new methodology was investigated to incorporate new USGS based Areal Reduction Factors (ARF) to replace the TP-40 based methodology hard-wired into HEC-1 and HEC-HMS. This paper presents a methodology to apply ellipsoidal areal reduction factors to the 24-hour storm used in the SAR basin tributary models.

The Upper San Antonio River Basin Test

The original LMMP study model (HEC-1) that is currently being used in the DFIRM process was selected for testing the impact of the adjusted rainfall rates. The watershed is approximately 134.5 square miles and the SAR USGS gauge at Loop 410 was selected as the point of interest to allow comparison with a gauged event. The accepted LMMP study model (HEC-1) produced a flow of 86,437 cfs using the rainfall rates supplied by the City of San Antonio Unified Development Code (UDC). For testing purposes, the model was updated with only the new precipitation. The updated model produced a flow of 106,917 cfs for the 1% annual chance event, which included TP-40 areal reduction by HEC-1. This

BEXAR COUNTY MAP MODERNIZATION

ARF STORM HYETOGRAPH METHOD

flow was considered unreasonable since the highest recorded flow at Loop 410 was 79,400 cfs during the October 1998 flood.

As part of the analysis of the SAR below the confluence with the Medina River, the ellipsoidal rainfall distribution method was applied by intersecting ellipsoid patterns provided by PBS&J with the SAR basin delineation in GIS to obtain a single weighted areal reduction factor for each major basin. The 330 Degree pattern produced the highest average rainfall for the major watersheds. The 1% annual chance frequency rainfall depths were distributed via a unit hyetograph to produce incremental hyetographs for each return period. The hyetographs were entered into HEC-1 and the model was run. The 1% annual chance flow at Loop 410 was reduced to 67,430 cfs, a 22% decrease from the accepted LMMP model. This is consistent with the 74.5 percent ARF suggested by the PBS&J report. Recognizing that the new rainfall dataset would most likely produce excessive flows throughout the study area, an alternate method was required to produce consistent flows as area increases down basin.

Areal Reduction Using the Frequency Storm Meteorological Model

Areal reduction in the tributary sub-basins using HMS was performed by setting storm size threshold values in the HMS meteorological model (Figure 1). The threshold values were determined by evaluating points of interest within each watershed model. Multiple threshold values were run and the results for each were exported to a spreadsheet provided by PBS&J (Figure 2). Table 1 contains selected model results from the current study Martinez B model. This approach parallels the areal reduction approach built into HEC-1 but does not create a hydrograph at every computation point in the model.

BEXAR COUNTY MAP MODERNIZATION
ARF STORM HYETOGRAPH METHOD

HMS * Meteorologic Model

File Edit Help

Meteorologic Model: 100-YR Subbasin List

Description: 100 YEAR ...

Precipitation Evapotranspiration

Method: Frequency Storm

Exceedance Probability: 1 %

Series Type: Annual

Max Intensity Duration: 5 Mins

Storm Duration: 24 Hr.

Peak Center: 50%

Storm Area (sq. mi.) 16

Duration	Precip Depth (in)
5 minutes	1.13
15 minutes	2.5
1 hour	4.35
2 hours	5.8
3 hours	6.6
6 hours	7.5
12 hours	8.8
24 hours	10.0
2 days	
4 days	
7 days	
10 days	

OK Apply Cancel

Meteorologic Model Description

Figure 1 - HMS Model With Reduction Threshold Set At 16 Miles

BEXAR COUNTY MAP MODERNIZATION ARF STORM HYETOGRAPH METHOD

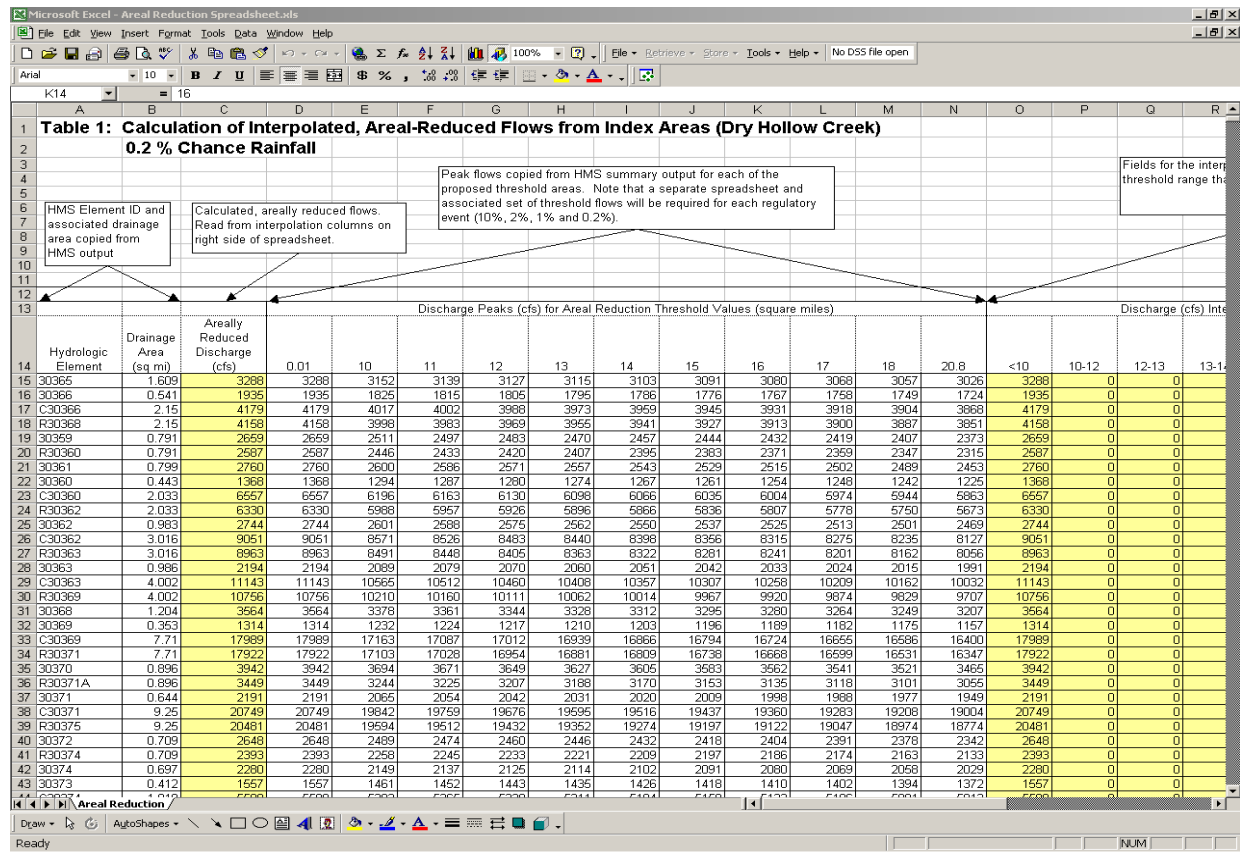


Figure 2 - Areal Reduction Spreadsheet Example.

Watershed	Model Point	Drainage Area	FIS	Current Study
E. Fk E. Br. Salitrillo	30109	0.93 mi ²	1,100 cfs	1,909 cfs
E. Br. Salitrillo	30110	1.3 mi ²	2,750 cfs	2,830 cfs
E. Salitrillo	30101	1.0 mi ²	2,851 cfs	3,125 cfs
	C30103	1.6 mi ²	3,126 cfs	4,968 cfs
	C30108	5.5 mi ²	5,820 cfs	7,639 cfs
	C30113	12.9 mi ²	10,740 cfs	14,107 cfs
Martinez Cr. B	C30162	9.5 mi ²	2,600 cfs	4,775 cfs
	C30170	14.0 mi ²	8,400 cfs	10,546 cfs
W. Salitrillo	C30116	2.2 mi ²	5,080 cfs	5,120 cfs
	C30118	3.8 mi ²	1,580 cfs	2,327 cfs

Table 1 – Selected Martinez B 1% Annual Chance Model Results vs. Existing FIS.

BEXAR COUNTY MAP MODERNIZATION ARF STORM HYETOGRAPH METHOD

The interest in testing the ellipsoid areal reduction factors on the SAR tributaries centers around the asymptotic limit of TP-40 areal reduction for 24-hour storms, which is around 8%. The general consensus believes that an 8% reduction is too conservative and a greater reduction should be applied in this region of Texas (USGS WRI 99-4267). Application of the full HMR-52 ellipsoid areal reduction method to the SAR tributaries is neither practical nor recommended, since most are fairly small. This investigation was conducted to test alternative methods to reduce flows in the SAR tributaries using ellipsoid areal reduction factors without the GIS procedure.

Alternative 1: Areal Reduction Using IDF Equations

The RWMS recommends a 24-hour storm with a 6-minute computation interval for SARB watershed models. PBS&J-2005 contains a letter dated March 3, 2005 that mentions the development of 24-hr, 2-day, 4-day, and 7-day unit hyetographs; however, Pape-Dawson only received the 2 to 7-day hyetographs and the tributary models use a 24-hr storm. Several approaches were examined to scale the 2-day hyetograph to 24-hours, but they were abandoned since IDF equations are non-linear and time is one of the variables. It became evident that the most prudent approach was to develop a new unit hyetograph for the 24-hour storm.

The Bexar County TXDOT IDF equation was selected as the starting point of development. The equation was used to calculate 24-hr, 15-min Depth-Duration-Frequency (DDF) curves for the 10, 4, 2, 1, and 0.2 percent annual chance storms. The alternating block method was used to build incremental and cumulative hyetographs for each storm. The cumulative hyetographs are shown in Figure 3. Since the curves are very close, they were averaged and the average hyetograph was normalized to produce a 24-hr, 15-minute unit hyetograph.

BEXAR COUNTY MAP MODERNIZATION

ARF STORM HYETOGRAPH METHOD

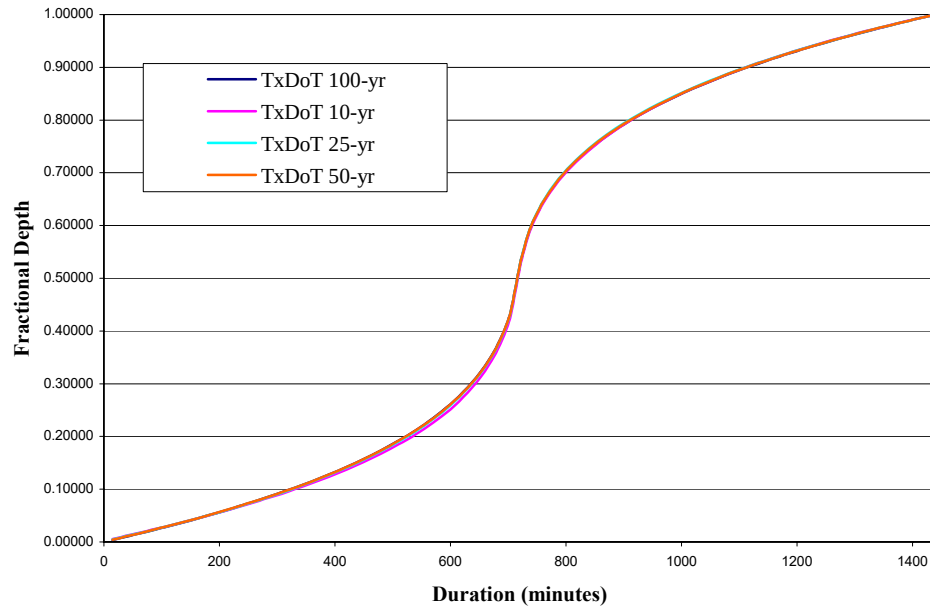


Figure 3 – Cumulative Hyetographs For Bexar County.

A 2-day unit hyetograph was also developed to compare with the PBS&J version (Figure 4). In order to maintain an equivalent peak rainfall duration, the Bexar County IDF equation was adjusted to more-or-less match the shape of the PBS&J unit 2-day hyetograph.

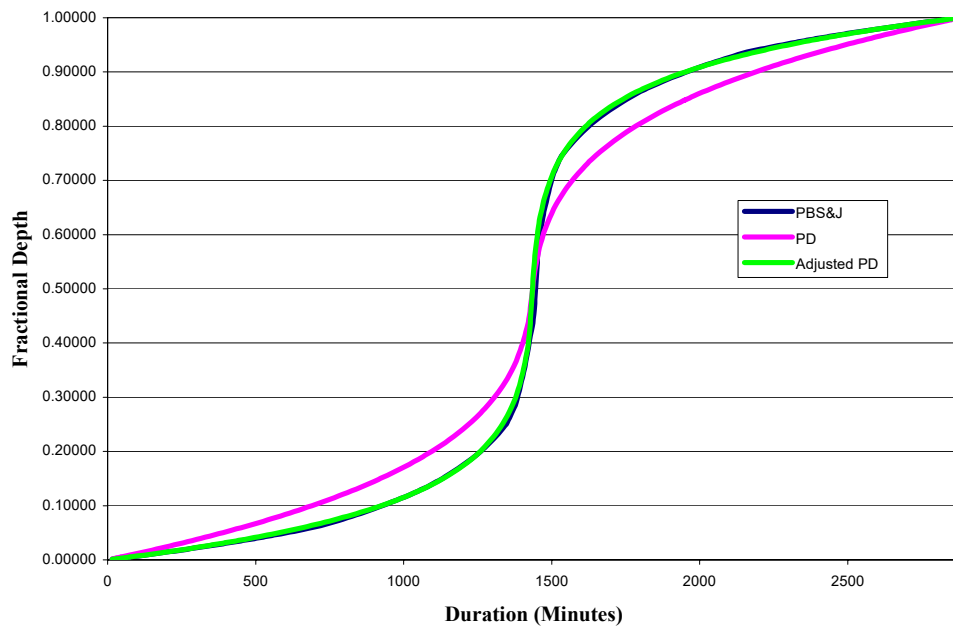


Figure 4 – Pape-Dawson Curves vs. PBS&J Curve.

BEXAR COUNTY MAP MODERNIZATION ARF STORM HYETOGRAPH METHOD

A new 24-hr, 15-minute unit hyetograph was then produced with the adjusted equation and imported into DSS using the new Excel DSS Add-in from the HEC. Once the unit hyetograph was in DSS, the areal reduction factors for ellipses (Table 7 of PBS&J-2005) were applied to get a design hyetograph for each storm and reduction threshold value. The ellipsoid areal reduction factors were interpolated by applying a smooth graph line in Excel and reading them from the graph. This proved to be more accurate than linear interpolation as Figure 5 shows. The new hyetographs were entered into HMS as gauges, the user-defined

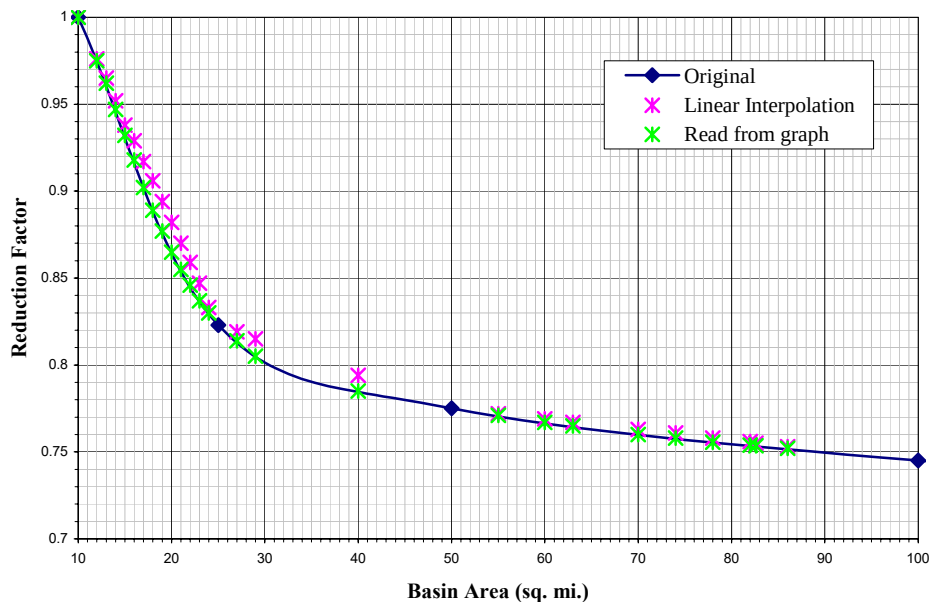


Figure 5 - ARF Interpolation

hyetograph meteorological model was applied, the model was run, and the output was entered into the reduction spreadsheet to obtain reduced flows. Table 2 compares the results between the current study model and the IDF-based model for the Martinez B drainage basin. The flow reductions were fairly severe across the board. The flows from basins of

BEXAR COUNTY MAP MODERNIZATION
ARF STORM HYETOGRAPH METHOD

Watershed	Model Point	Drainage Area	Current 1% chance	15-min PD 1% chance	% Diff.
E. Fk E. Br. Salitrillo	30109	0.93 mi ²	1,909 cfs	1,482 cfs	-29 %
E. Br. Salitrillo	30110	1.3 mi ²	2,830 cfs	2,171 cfs	-30 %
E. Salitrillo	30101	1.0 mi ²	3,125 cfs	2,149 cfs	-45 %
	C30103	1.6 mi ²	4,968 cfs	3,441 cfs	-44 %
	C30104	1.7 mi ²	5,062 cfs	3,509 cfs	-44 %
	C30108	5.5 mi ²	7,639 cfs	6,033 cfs	-27 %
	C30113	12.9 mi ²	14,107 cfs	8,588 cfs	-64 %
Martinez Cr. B	C30162	9.5 mi ²	4,775 cfs	3,745 cfs	-28 %
	C30170	14.0 mi ²	10,546 cfs	7,567 cfs	-39 %
W. Salitrillo	C30116	2.2 mi ²	5,120 cfs	3,857 cfs	-33 %
	C30118	3.8 mi ²	2,327 cfs	1,842 cfs	-26 %

Table 2 – Comparison Of HMS Model Output For Alternative 1.

10 mi² or less should have matched, or been very close to, the current study model since the ellipsoid areal reduction factor is 1.0 for 10 mi² or less. An examination of the model output revealed that HMS equally distributed the 15-min peak rainfall total of 1.41 inches over three 6-min intervals. This functionally decreased the rainfall depth, causing a respective decrease in runoff. To get a more valid comparison, a 6-minute unit hyetograph was developed from the same IDF equation to match the model computation interval, but DSS would not accept a 6-minute dataset from any source outside of HMS. It was necessary to change the recommended computation interval from 6 minutes to 5 minutes to get the new hyetographs into DSS. After the necessary changes were made, a new model run was performed. The comparisons from that run are shown in Table 3. The decreased time interval clearly made a

BEXAR COUNTY MAP MODERNIZATION

ARF STORM HYETOGRAPH METHOD

Watershed	Model Point	Drainage Area	Current 1% chance	5-min PD 1% chance	% Diff.
E. Fk E. Br. Salitrillo	30109	0.93 mi ²	1,909 cfs	1,645 cfs	-16%
E. Br. Salitrillo	30110	1.3 mi ²	2,830 cfs	2,428 cfs	-17%
E. Salitrillo	30101	1.0 mi ²	3,125 cfs	2,535 cfs	-23 %
	C30103	1.6 mi ²	4,968 cfs	4,021 cfs	-24 %
	C30104	1.7 mi ²	5,062 cfs	4,116 cfs	-23 %
	C30108	5.5 mi ²	7,639 cfs	6,652 cfs	-15 %
	C30113	12.9 mi ²	14,107 cfs	12,437 cfs	-13 %
Martinez Cr. B	C30162	9.5 mi ²	4,775 cfs	4,150 cfs	-15 %
	C30170	14.0 mi ²	10,546 cfs	9,093 cfs	-16 %
W. Salitrillo	C30116	2.2 mi ²	5,120 cfs	4,354 cfs	-18 %
	C30118	3.8 mi ²	2,327 cfs	2,025 cfs	-15 %

Table 3 – Comparison Of HMS Model Output Using A 5-Minute Hyetograph.

significant improvement, but the results were still not within an acceptable range of the current study model because the maximum 1% annual chance, 5-minute precipitation depth was only 0.75 inches. The new rainfall data provided by PBS&J set the 1% chance, 5-min rainfall total at 1.13 inches.

Alternative 2: Areal Reduction Using HMS Hyetographs

The second alternative was a slight modification to the first. Instead of creating new hyetographs from IDF curves, the hyetographs were extracted from HMS frequency-storm runs with the storm size set at 0.01 square miles. This was done to ensure that the unreduced peak rainfall totals matched the HMS frequency-storm peak totals for each storm. That way, the flow from the smaller drainage basins would match the current study model and the larger basins would see a reduction. The hyetographs were put into Excel, normalized and

BEXAR COUNTY MAP MODERNIZATION

ARF STORM HYETOGRAPH METHOD

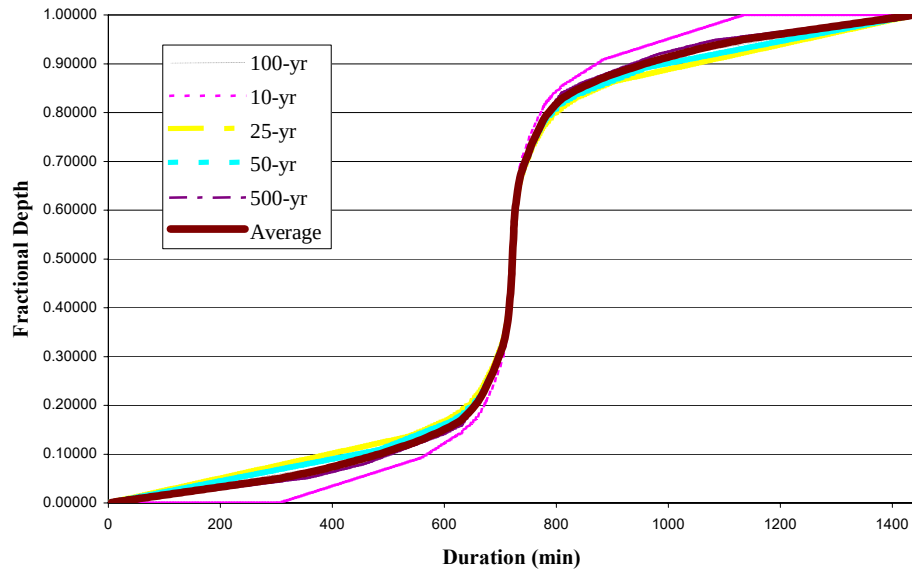


Figure 6 - Unit Frequency Storm Hyetographs From HMS.

plotted (Figure 6). An average unit hyetograph was calculated, the new 1% annual chance, 24-hr rainfall totals were distributed over it, and the peak 5-minute values were compared. The results are listed in Table 4. The differences were significant enough to justify dropping

Frequency	HMS	Unit Hyetograph	% Difference
10%	0.78 in	0.71 in	-9.9
4%	0.93 in	0.89 in	-4.5
2%	1.04 in	1.07 in	2.8
1%	1.13 in	1.19 in	5.0
0.2%	1.52 in	1.63 in	6.8

Table 4 - Peak Rainfall Total Comparison.

the unit hyetograph approach and applying the ellipsoid areal reduction factors to the individual HMS frequency storm hyetographs. The extracted hyetographs were written to

BEXAR COUNTY MAP MODERNIZATION

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DSS from Excel as the 10-mile hyetographs for each frequency. The reduction factors for each threshold value were applied to the 10-mile hyetograph and entered into DSS. The HMS model was modified to accept the new hyetographs and was run. Table 5 compares the

Watershed	Model Point	Drainage Area	Current 1% chance	HMS 1% chance	% Diff.
E. Fk E. Br. Salitrillo	30109	0.93 mi ²	1,909 cfs	1,925 cfs	1%
E. Br. Salitrillo	30110	1.3 mi ²	2,830 cfs	2,856 cfs	1%
E. Salitrillo	30101	1.0 mi ²	3,125 cfs	3,175 cfs	2%
	C30103	1.6 mi ²	4,968 cfs	5,048 cfs	2%
	C30104	1.7 mi ²	5,062 cfs	5,154 cfs	2%
	C30108	5.5 mi ²	7,639 cfs	7,714 cfs	1%
	C30113	12.9 mi ²	14,107 cfs	14,377 cfs	2%
Martinez Cr. B	C30162	9.5 mi ²	4,775 cfs	4,832 cfs	1%
	C30170	14.0 mi ²	10,546 cfs	10,622 cfs	1%
W. Salitrillo	C30116	2.2 mi ²	5,120 cfs	5,202 cfs	2%
	C30118	3.8 mi ²	2,327 cfs	2,345 cfs	1%

Table 5 – Comparison Of HMS Model Output For Alternative 2.

Alternative 2 output with the current study model. The 10 mi² flows were within 1-2% of the frequency-storm model 10 mi² flows and the flows for drainage areas greater than 10 mi² were reduced to reasonable values, compared to previous model runs. Figure 7 shows a comparison of all model points between the two models. The ellipsoid areal reduction factors applied to the HMS frequency storm hyetographs produced the desired results. The flows out

BEXAR COUNTY MAP MODERNIZATION ARF STORM HYETOGRAPH METHOD

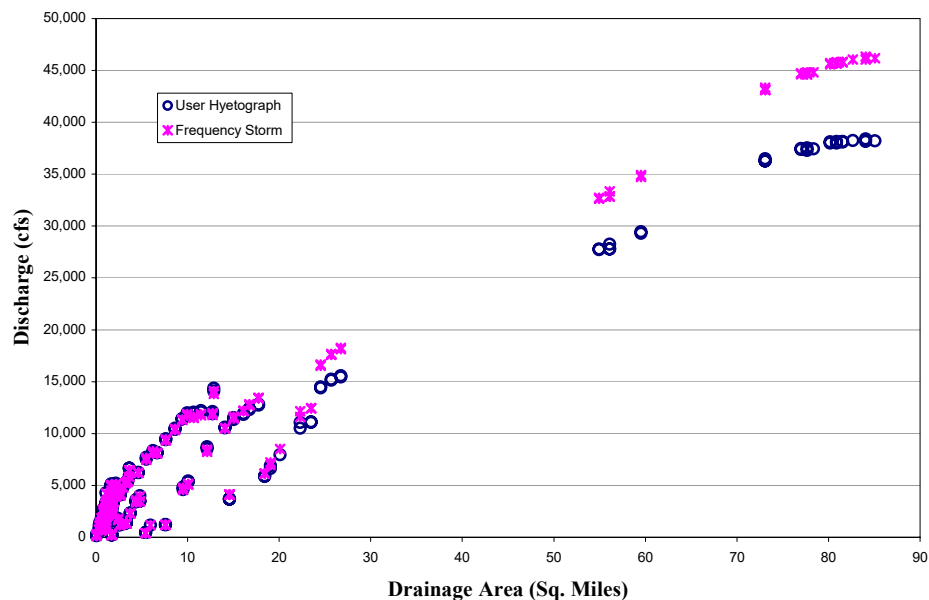


Figure 7 - Comparison Of Current Model With Alternative 2 Model.

of drainage basins 10 mi² or less were not reduced and received the full point rainfall. The flows out of drainage basins larger than 10 mi² were reduced in proportion to their size. The maximum flow reduction was 21%, which reasonably agrees with the 22% reduction observed with the LMMP model test. The Alternative 2 method is proposed for use to perform areal reduction that is consistent with the new rainfall data that has been adopted.

Conclusions

The approach described in this report provides a workable solution to incorporating the USGS adjusted rainfall rates into the RWMS while maintaining consistency with existing local standards. This approach has the advantages of allowing the conversion of legacy HEC-1 models to HMS and standardizing the hydrologic methods across the entire study area.

San Antonio River Authority
White Paper – Modify Hydrology Models
Rev. 2

Process to Obtain Peak Discharge Data and Update or Modify Hydrology Models

The purpose of this document is to outline the process of obtaining peak discharge data and modifying/updating the Digital Flood Insurance Rate Map (DFIRM) hydrology models. Over the period in time in which the hydrology models were developed for the San Antonio River Basin (SARB), several different HEC-HMS versions were used (2.2.2, 3.0 and 3.1.0). This outline is based on HEC-HMS version 3.1.0 since it is the most current version and is compatible with the previous versions.

Items that are necessary to obtain peak discharges and update/modify the DFRIM hydrology models include:

- Complete HEC-HMS model of the watershed of interest saved on local hard drive
- Copy of the "Calculation of Interpolated Areal-Reduced Flows from Index Areas" Excel spreadsheet
- Copy of the San Antonio River Basin Regional Modeling Standards for Hydrology and Hydraulic Modeling
- Copy of the hydrology Technical Support Data Notebook (TSDN) that includes in watershed of interest

All of these items can be found on the Bexar Regional Watershed Management website located at www.sara-gis-tx.org/website/dfirm/.

Obtaining Peak Discharge Data

The quickest way to find peak discharges used for the DFIRM is through the watershed-specific hydrology TSDN. The TSDN includes a watershed schematic map in the appendices. The map shows the calculation node and sub-watershed structure of the HEC-HMS model. Each calculation node is identified by an alpha-numeric label. The labels are typically coded in a way that clearly identifies which sub-watershed(s) are associated with the nodes. Peak discharges for the different frequency storm events for each node can be found in the aerial reduction

spreadsheet once the node(s) of interest are identified using the schematic map. Discharges can be obtained from the spreadsheet without having to rerun the model under most circumstances.

If the HEC-HMS model must be rerun to obtain peak discharge data, the following paragraphs describe the process to obtain the correct flows. It must be noted that the flow results of any single model run cannot be used because of the areal reduction method employed for the DFIRM project. The flows must be taken from the "Calculation of Interpolated, Areal-Reduced Flows from Index Areas" Excel spreadsheet. Therefore, one must save a copy of the model and the areal reduction spreadsheet for the model being used on the local hard drive; furthermore, the "Read Only" property of the spreadsheet and model files must be cleared before proceeding. The spreadsheet can be found on the CD-ROM/DVD located in the appendix of the TSDN. Furthermore, to avoid any path issues, the models should be saved in C:\hmsproj*model name*\. Some models use gridded precipitation files and any changes in the path will require the user to manually re-establish all of the paths for the DSS and grid cell parameter files.

Once the "Read Only" property has been cleared, open the model in HEC-HMS. If a warning or error message pops up that says data was not found or data was not able to be opened as indicated, the file paths of the support files that make up a HEC-HMS model will require editing or the support files will need to be placed in the same file structure. The paths can be changed by opening the individual model files using a text editor such as Notepad or WordPad.

A HEC-HMS model contains three components. They are the basin model, the meteorological model, and the control specification. After the HEC-HMS project is open, a basin model can be viewed by selecting the basin name under the "Basin Models" directory. Normally there are several basin models that represent different scenarios such as existing conditions, future conditions, proposed conditions, and possibly calibration; however, the model scenarios could have been developed as separate projects just as well. Figure 2 shows the HEC-HMS project menu screen for Leon Creek as an example.

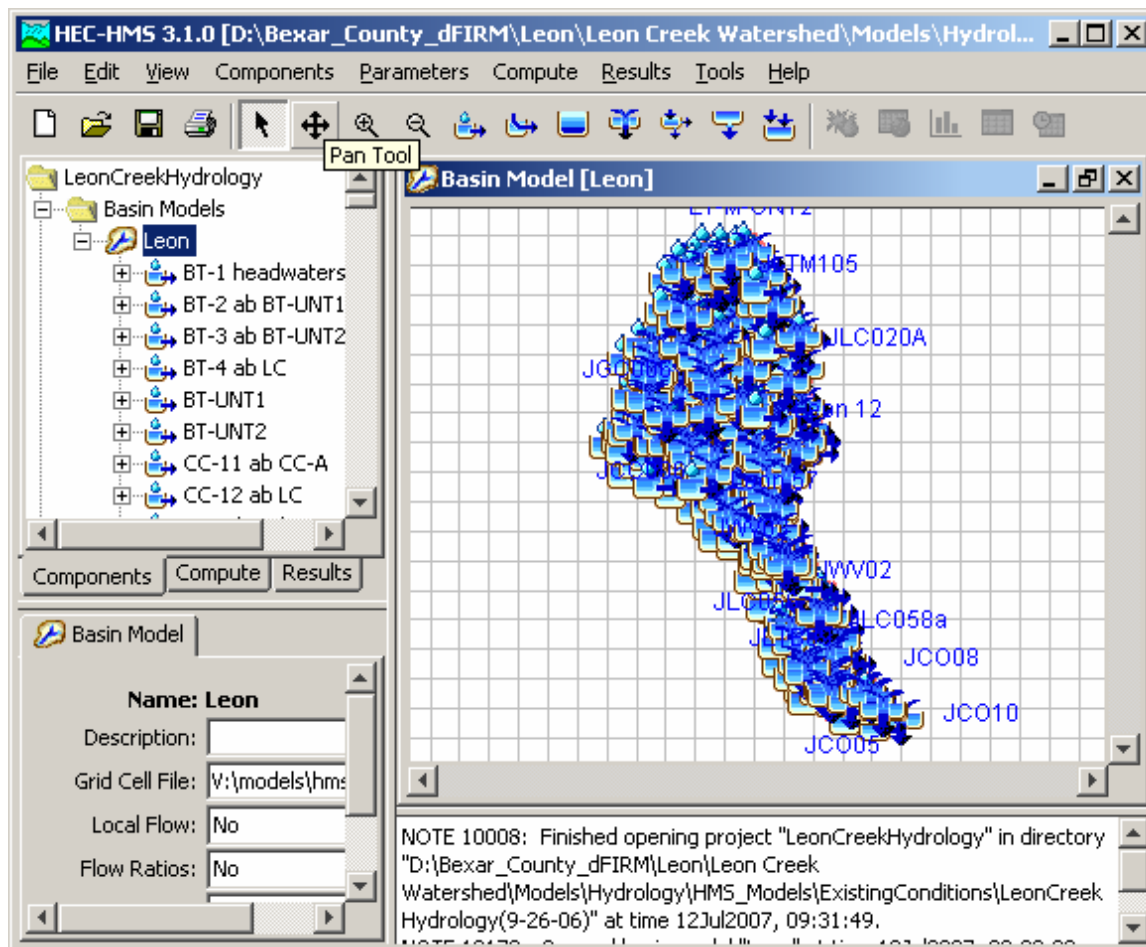


Figure 2

Selecting the basin name will bring up the model schematic, which is comprised of junction nodes, routing reaches and sub-watersheds. The parameters of a model element can be viewed by selecting the element ID and looking in the pane on the bottom left.

The configured model runs are located on the "Compute" tab under the "Simulation Run" folder (see Figure 3). How the desired discharges are obtained will depend on the rainfall simulation methodology employed for the watershed of interest. Point rainfall, ellipsoidal, point/ellipsoidal rainfall combination and the frequency storm are four possible rainfall simulation methodologies used in the hydrologic analysis of watersheds within Bexar, Wilson, Karnes and Goliad counties.

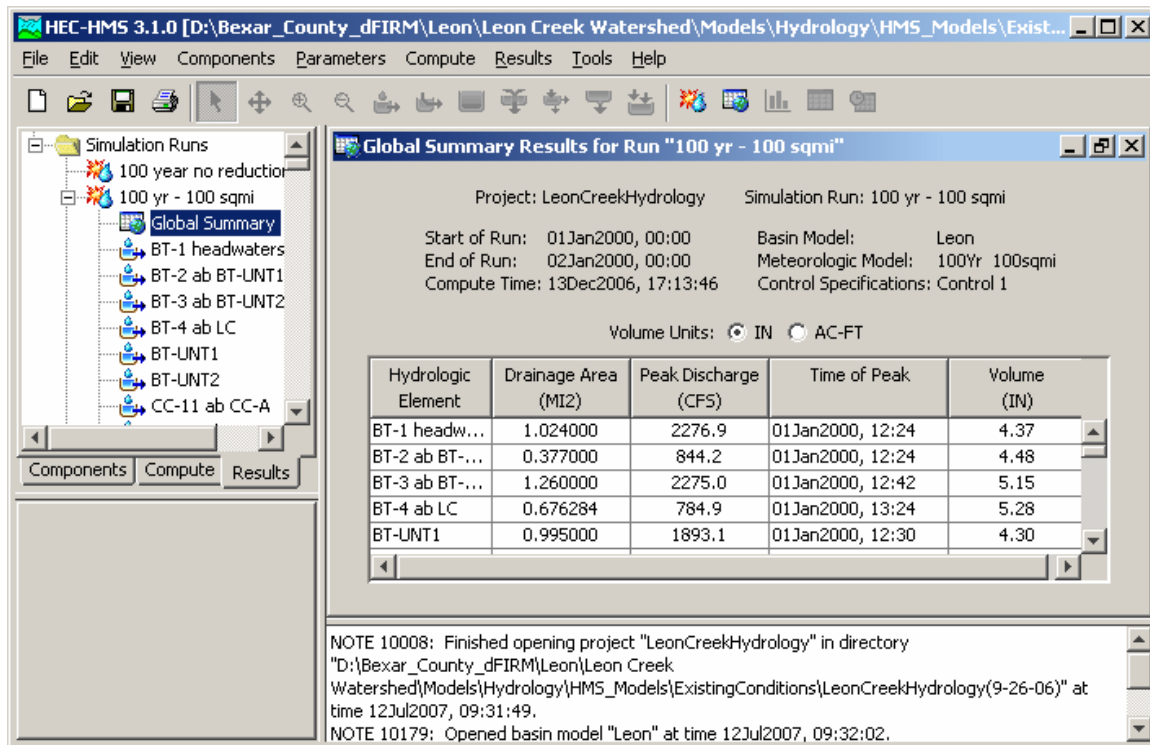


Figure 3

If the point rainfall methodology was used, several simulation runs for each storm frequency will need to be performed. The different model runs are necessary to convert the point rainfall into area-averaged rainfall (including areal reduction) for sub-watersheds larger than 10 square miles. All simulations need to be run for all locations within the chosen watershed. To perform a model run, simply highlight the appropriate run, right click, and select "Compute." For the Leon Creek example, the user would have to perform seven model runs to obtain the 100-year peak discharge data for any given calculation node.

After computing all simulations, select the "Results" tab and chose one of the simulations for the desired storm frequency. Select the "Global Summary" and copy the "Discharge Peak," "Hydrologic Element" and "Basin Area" columns individually (see Figure 3) and paste them in the appropriate columns in the areal reduction spreadsheet (Figure 4). Each model run applies a different areal reduction factor for index storm sizes selected by the original modelers. Therefore, the discharge values must be inserted into the column that corresponds to the model run's storm size, i.e. 100 sq mi.

Microsoft Excel - Areal Reduction Spreadsheet.xls

File Edit View Insert Format Tools Data Window Help

100% No DSS File open

Table 1: Calculation of Interpolated, Areal-Reduced Flows from Index Areas (Dry Hollow Creek)
0.2 % Chance Rainfall

Peak flows copied from HMS summary output for each of the proposed threshold areas. Note that a separate spreadsheet and associated set of threshold flows will be required for each regulatory event (10%, 2%, 1% and 0.2%).

Fields for the interpolation range threshold

HVS Element ID and associated drainage area copied from HVS output

Calculated, areally reduced flows. Read from interpolation columns on right side of spreadsheet.

Discharge Peaks (cfs) for Areal Reduction Threshold Values (square miles)

Hydrologic Element	Drainage Area (sq mi)	Areal Reduced Discharge (cfs)	0.01	10	11	12	13	14	15	16	17	18	20.8	<10	10-12	12-13	13-14
30365	1.609	3288	3288	3152	3139	3127	3115	3103	3091	3080	3068	3057	3026	3288	0	0	0
30366	0.541	1935	1935	1825	1815	1805	1795	1786	1776	1767	1758	1749	1724	1935	0	0	0
30366	2.15	4179	4179	4017	4002	3988	3973	3959	3945	3931	3918	3904	3868	4179	0	0	0
R30368	2.15	4158	4158	3998	3983	3969	3955	3941	3927	3913	3900	3887	3851	4158	0	0	0
30369	0.791	2659	2659	2511	2497	2483	2470	2457	2444	2432	2419	2407	2373	2659	0	0	0
R30360	0.791	2587	2587	2446	2433	2420	2407	2395	2383	2371	2359	2347	2315	2587	0	0	0
30361	0.799	2760	2760	2600	2586	2571	2557	2543	2529	2515	2502	2489	2453	2760	0	0	0
30360	0.443	1368	1368	1294	1287	1280	1274	1267	1261	1254	1248	1242	1225	1368	0	0	0
R30360	2.033	6557	6557	6196	6163	6130	6098	6066	6035	6004	5974	5944	5863	6557	0	0	0
R30362	2.033	6330	6330	5988	5957	5926	5896	5866	5836	5807	5778	5750	5673	6330	0	0	0
30362	0.983	2744	2744	2601	2588	2575	2562	2550	2537	2525	2513	2501	2469	2744	0	0	0
R30362	3.016	9051	9051	8571	8526	8483	8440	8398	8356	8315	8275	8235	8127	9051	0	0	0
R30363	3.016	8963	8963	8491	8446	8405	8363	8322	8281	8241	8201	8162	8056	8963	0	0	0
30363	0.986	2194	2194	2089	2079	2070	2060	2051	2042	2033	2024	2015	1991	2194	0	0	0
R30363	4.002	11143	11143	10565	10512	10460	10408	10357	10307	10258	10209	10162	10032	11143	0	0	0
R30369	4.002	10756	10756	10210	10160	10111	10062	10014	9967	9920	9874	9829	9707	10756	0	0	0
30368	1.204	3564	3564	3378	3361	3344	3328	3312	3296	3280	3264	3249	3207	3564	0	0	0
R30368	0.353	1314	1314	1232	1224	1217	1210	1203	1196	1189	1182	1175	1157	1314	0	0	0
R30369	7.71	17989	17989	17163	17087	17012	16939	16866	16794	16724	16655	16586	16400	17989	0	0	0
R30371	7.71	17922	17922	17103	17028	16954	16881	16809	16738	16668	16599	16531	16347	17922	0	0	0
30370	0.896	3449	3449	3244	3225	3207	3188	3170	3153	3136	3118	3101	3065	3449	0	0	0
R30371A	0.896	3449	3449	3244	3225	3207	3188	3170	3153	3136	3118	3101	3065	3449	0	0	0
30371	0.644	2191	2191	2065	2054	2042	2031	2020	2009	1998	1988	1977	1949	2191	0	0	0
R30371	9.25	20749	20749	19842	19759	19676	19595	19516	19437	19360	19283	19208	19004	20749	0	0	0
R30375	9.25	20481	20481	19594	19512	19432	19352	19274	19197	19122	19047	18974	18774	20481	0	0	0
30372	0.709	2393	2393	2258	2245	2233	2221	2209	2197	2186	2174	2163	2133	2393	0	0	0
R30374	0.697	2280	2280	2149	2137	2125	2114	2102	2091	2080	2069	2058	2029	2280	0	0	0
30373	0.412	1557	1557	1461	1452	1443	1435	1426	1418	1410	1402	1394	1372	1557	0	0	0

Areal Reduction

Figure 4

This process must be repeated for all simulations for a given frequency in order to obtain the peak discharges for the entire watershed. Note that only the peak discharge information will need to be copied from the HEC-HMS model to the areal reduction spreadsheet for the remaining simulations. After this process is complete, the peak discharge for a given node can be found under the column titled "Areal Reduced Discharge" within the excel spreadsheet.

If the ellipsoidal methodology is used, several simulations for a given storm frequency may exist due to the optimal storm positioning that is necessary to produce peak discharge results along a creek or river system. In order to determine which simulations produces peak discharges at the different nodal locations, it is necessary to refer to documentation within the model or the hydrology TSDN. Once this is determined, the peak discharge information can be accessed directly by selecting the 'Global Summary' for the appropriate simulation under the 'Results' tab (after running the simulation). Note that this peak discharge information is only

applicable for the specific node in which the simulation is intended. Since the area reduction has been included in the simulation itself, there is no need to interpolate the results as is necessary with the point rainfall methodology.

The point /ellipsoidal rainfall methodology is only utilized to obtain peak discharge rates for the Lower San Antonio River. The storm simulation titled “Wilson County centered over Wilson, Karnes and Goliad counties” was the storm simulation chosen to produce appropriate peak discharge results. As with the ellipsoidal rainfall methodology, peak discharge results can be accessed directly from the model without interpolation after running the simulation. Also, this single storm simulation provides correct peak discharge information at all nodal locations along the Lower San Antonio River (unlike the ellipsoidal rainfall methodology).

The frequency storm methodology is only utilized to obtain peak discharge rates for the Upper Cibolo Creek. All area reductions are performed automatically during the HEC-HMS model simulation. Peak discharge information for all nodal locations along the creek can be accessed directly through a single storm simulation within the model.

The hydrology TSDN details which storm simulation methodology is used for a particular watershed and contains additional information on each methodology.

Modifying/ Updating the DFIRM Model

After a hydrology model has been developed, it must be maintained to reflect the continual changes of a development within the watershed. An important tool for the development and modification of the DFIRM hydrology models is ArcGIS (Figure 5). Much of the parameter input used in hydrology modeling can be spatially related and calculated in ArcGIS. Once parameter changes have been determined using the many tools available in ArcGIS, they can be easily updated in the hydrology model.

Included in the appendix of the hydrology TSDN is a CD-ROM/DVD containing shapefiles that contain HEC-HMS parameter data for each watershed. This spatial parameter data includes curve number, impervious cover and sub-basin area. These shapefiles can be modified to reflect watershed changes. After modifications are complete, HEC-HMS parameter inputs can be recalculated. Other HEC-HMS parameter input not formatted for ArcGIS must be calculated by other means.

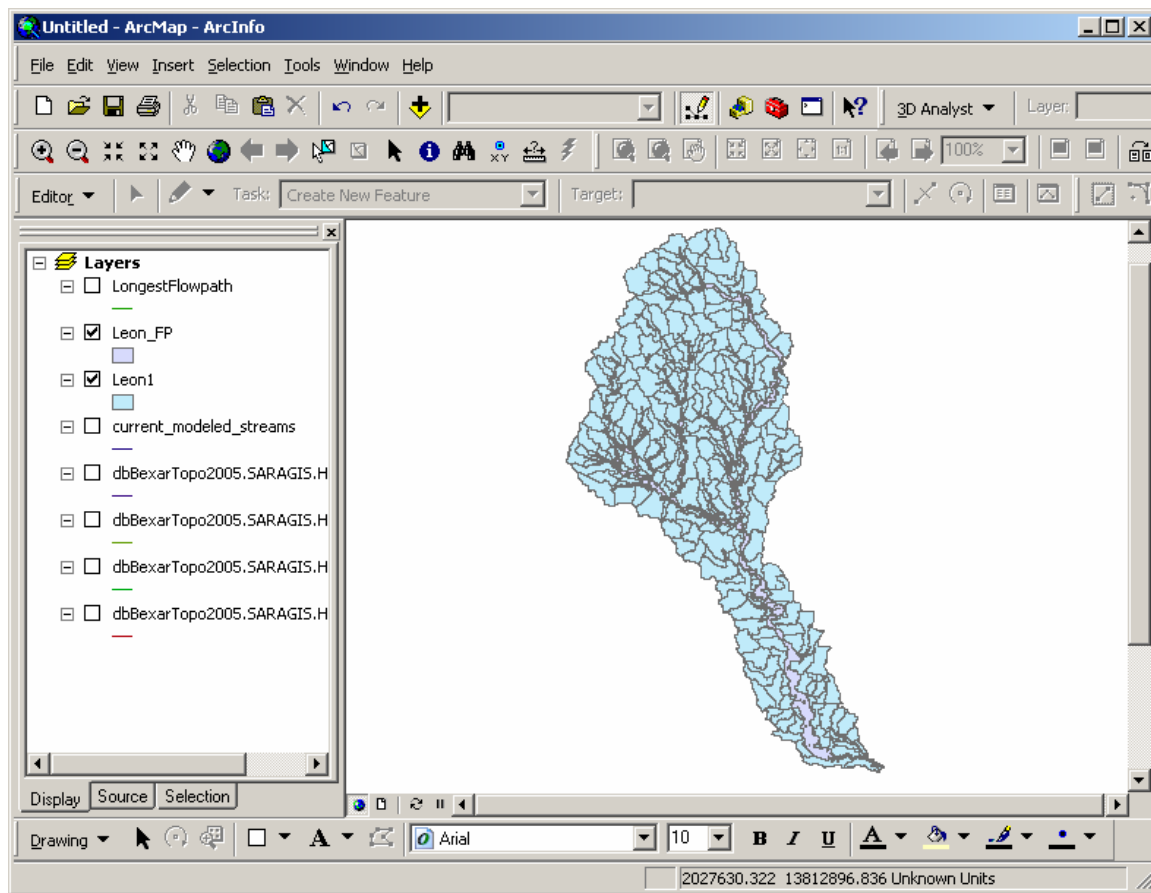


Figure 5

Parameter input can be updated within the HEC-HMS model by selecting the “Parameters” menu and choosing the appropriate category. In order to obtain the discharge results due to a parameter change, one will need to run all simulations for each storm frequency as outlined in the previous section.

This document generally describes the process of obtaining peak discharge data and modifying/updating the DFIRM hydrology models. Additional information pertaining to the methodology and parameters used to model the watersheds within Bexar, Wilson, Karnes and Goliad counties can be found in the watershed specific hydrology TSDN. Supplemental information relating to the use of HEC-HMS hydrologic modeling software can be found in the HEC-HMS User Manual and Technical Reference Manual.

APPENDIX E: GEOSPATIAL DATA GUIDELINES

E.1. COORDINATE SYSTEM AND DATUM

The standard coordinate system will be Texas State Plane, South Central Zone (in grid format). The standard horizontal datum will be North American Datum of 1983 (NAD83), and the standard vertical datum will be the North American Vertical Datum of 1988 (NAVD88). The standard unit will be feet.

• Projected Coordinate System:	NAD_1983_StatePlane_Texas_South_Central_FIPS_4204_Feet
• Projection:	Lambert_Conformal_Conic
• False_Easting:	1968500.00000000
• False_Northing:	13123333.33333333
• Central_Meridian:	-99.00000000
• Standard_Parallel_1:	28.38333333
• Standard_Parallel_2:	30.28333333
• Latitude_Of_Origin:	27.83333333
• Linear Unit:	Foot_US
• Geographic Coordinate System:	GCS_North_American_1983
• Datum:	D_North_American_1983
• Prime Meridian:	Greenwich
• Angular Unit:	Degree

E.2. SOFTWARE

Geospatial data submittals should be compatible with the ESRI software suite. Most CADD software is able to export layers into a suitable shapefile format. A variety of free software is available to facilitate the population of the data fields required in Section A.4. Examples include but are not limited to:

- Quantum GIS (<http://www.qgis.org>)
- Geographic Resources Analysis Support System (<http://grass.osgeo.org/>)
- System for Automated Geoscientific Analyses (<http://www.saga-gis.org/>).

Please note the availability of these resources may change without notice.

E.3. METADATA STANDARDS

In order to provide the greatest compatibility with standards already in use, facilitate data sharing efforts and take advantage of tried and true metadata specifications, it is recommended the use of the Federal Geographic Data Committee (FGDC) Content Standard for Digital Geospatial Metadata (FGDC-STD-001-1998) as its metadata standard.

The FGDC Metadata standard specifies data elements and groups of data elements to provide documentation for spatial data. It also specifies the order in which those elements appear in the metadata file as well as which elements are required (referred to as “core” elements) and which are considered optional.

For the purposes of this project, the FGDC required elements will be considered the minimum metadata requirements but it is strongly recommended that any available optional elements be included whenever possible. The FGDC required items are:

1. Description
2. Keywords
3. Citation
4. Extent
5. Time period of content
6. Status
7. Access and use constraints
8. Metadata contact
9. Horizontal accuracy statements.
10. Vertical accuracy statements.

The FGDC data elements and detailed specification can be found at:
http://www.fgdc.gov/metadata/documents/workbook_0501_bmk.pdf

E.4. TOPOLOGY

Care should be taken in the development of line and polygon geospatial data types to ensure feature edges and vertices snap to those of adjoining features. Common topology errors to avoid are provided in Table 1. When polygon shapefiles are prepared using CADD software, closed polylines should be used.

Table 1 - Common Polygon Topology Errors

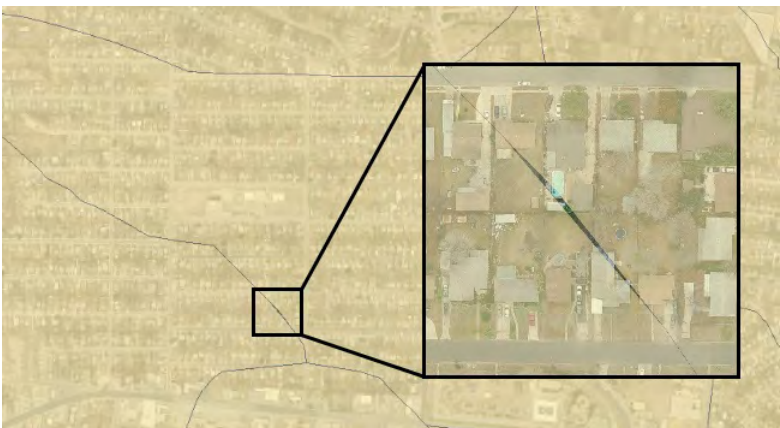
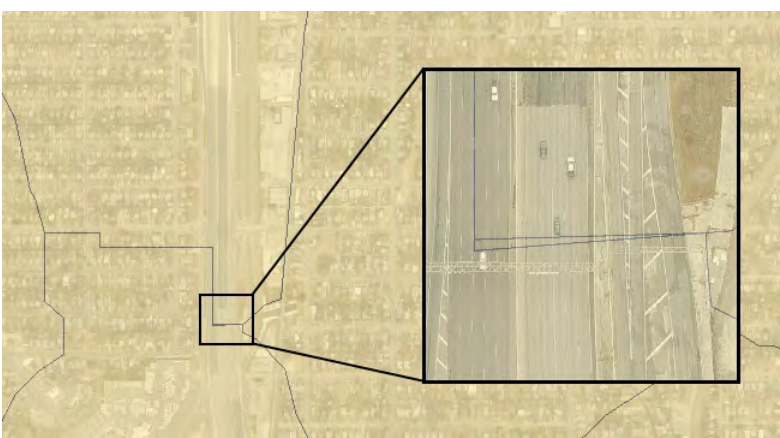
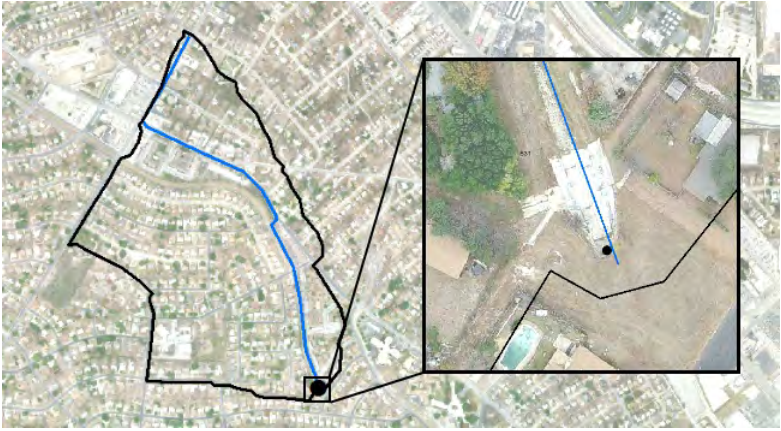
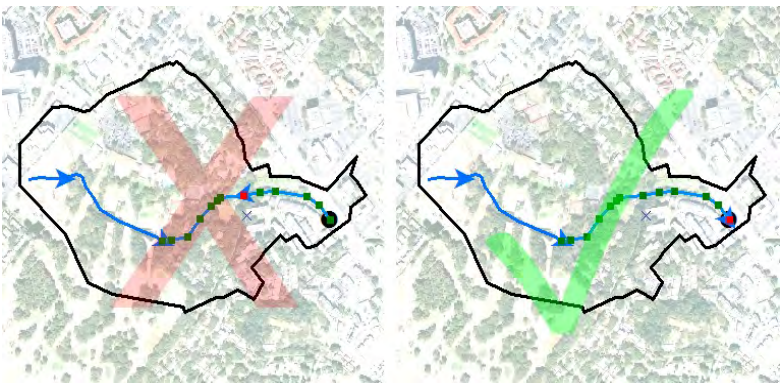
Overlap: one or more sides of a polygon extend over the boundary of an adjacent polygon.	
Gap: the sides of adjacent polygons do not touch.	
Sliver: a small polygon	

Table 2 - Common Polyline Topology Errors

Dangling Node: a node on a line or polygon feature that is supposed to intersect a point or a node on another feature but doesn't.	
Line Double-Back: a line segment drawn over itself	

E.5. SHAPEFILE CONTENT REQUIREMENTS

The tables in the next few pages present the feature attributes required for each of the shapefiles listed in the Geospatial Documentation Guidelines for Hydrology (Section 3.10.2), Hydraulics (Section 4.8.2), and Mapping (Section 6.3) which included:

- Base Flood Elevations (CLOMR/LOMR submittal)
- CLOMR/LOMR Boundaries (CLOMR/LOMR submittal)
- Cross Sections (Regional hydraulic model update)
- Floodplains (Polygon) (CLOMR/LOMR submittal)
- Floodplains (Line) (CLOMR/LOMR submittal)
- Junctions (Regional hydrology model update)
- Land Use (Regional hydrology model update)
- Reaches (Regional hydrology model update)
- Stream Centerlines (Regional hydraulic model update)
- Structures (Regional hydraulic model update)
- Sub-basins (Regional hydrology model update)

These files are only requested if the information is being modified as a result of the study. The data submitted does not need to extend beyond the study area.

The floodplain line features should match their corresponding floodplain polygon feature outlines. An example is provided in Figure 1. The floodplain polygon should have features covering the area inside the CLOMR/LOMR boundary, including Zone X for areas outside other floodplain zones.

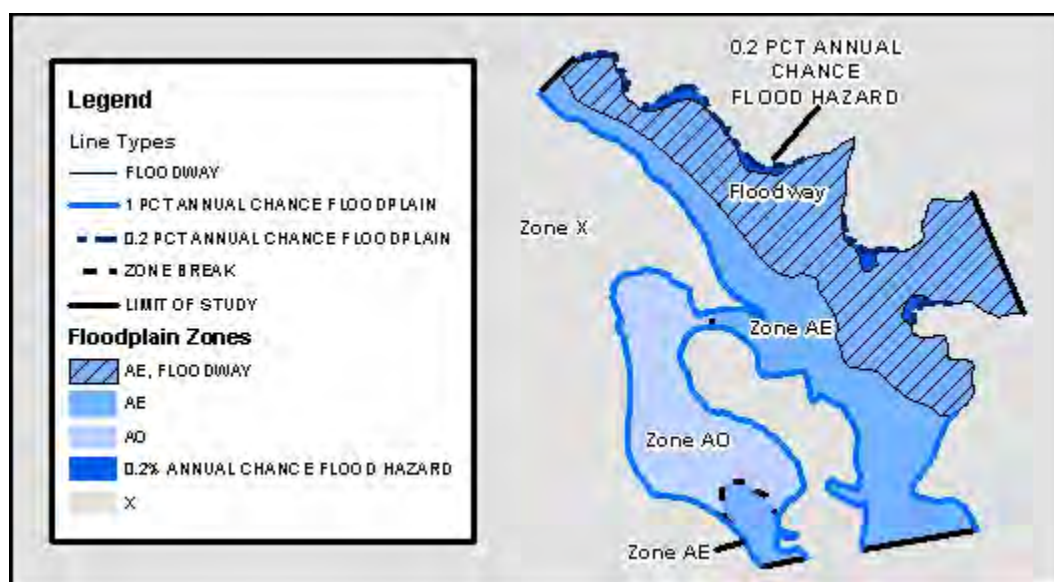


Figure 1 - Floodplain Zones and Line Types

Base Flood Elevations

Feature Name: S_BFE

Geometry Type: Polyline

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
BFE_LN_ID	Text	25		Unique feature identifier.	(Leave field empty)
ELEV	Double	Default	2	Base Flood Elevation. The rounded, whole elevation of the 1-percent-annual-chance flood.	
LEN_UNIT	Text	11		Measurement system used for the BFEs.	CENTIMETERS, FEET, INCHES, KILOMETERS, METERS, MILES, MILES
V_DATUM	Text	17		Vertical datum. The vertical datum indicates the reference surface from which the flood elevations are measured.	MSL, NAVD88, NGVD29, LOCAL TIDAL DATUM, MLLW, MLW
SOURCE_CIT	Text	11		(Leave field empty.)	(Leave field empty.)

CLOMR/LOMR Boundary

Feature Name: S_LOMR

Geometry Type: Polyline

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
DFIRM_ID	Text	15		Study Identifier. For a single-jurisdiction flood risk project, the value is composed of the two-digit State FIPS code and the four-digit FEMA CID code (e.g., 480001). For a countywide flood risk project, the value is composed of the two-digit State FIPS code, the three-digit county FIPS code, and the letter "C" (e.g., 48107C). Within each FIRM Database, the DFIRM_ID value will be identical.	(Same value as effective DFIRM.)
EFF_DATE	Date	Default	0	Effective Date. Effective date of the LOMR.	(Leave field empty.)
CASE_NO	Text	13		Case Number. This is the case number of the LOMR that is assigned by FEMA. The case number is used to track the LOMR's supporting documentation and will be generated following submittal.	(Leave field empty.)
SCALE	Text	5		Map Scale. This is the denominator of the effective LOMR scale as a ratio. For example, 24000 is the denominator for a 1" = 2000' map.	STANDARD 1"=500'/STANDARD 1"=1000'/STANDARD 1'=2000'
STATUS	Text	12		Status of the LOMR. This will be generated following submittal with one of the following values : SUPERSEDED, REVALIDATED, INCORPORATED, REDETERMINED, EFFECTIVE.	(Leave field empty.)
SOURCE_CIT	Text	11		(Leave field empty)	(Leave field empty.)

Cross Sections

Feature Name: S_XS

Geometry Type: Polyline

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID				An automatically generated unique identifier.	
XS_LN_ID	Text	25		Unique feature identifier.	(Leave field empty)
XS_LTR	Text	12		Cross Section Letter - The letter or number that is assigned to the cross section on the hardcopy FIRM and in the FIS report. This field is populated when the cross section is lettered in the Effective FEMA cross section.	(Same value as effective FEMA cross section or leave field empty)
START_ID	Text	25		Station Start Identification - Describes the origin for the measurements in the STREAM_STN field.	(Same value as adjacent effective FEMA cross section or leave field empty)
STREAM_STN	Double	Default		Stream Station - The measurement along the stream centerline or profile baseline to the cross section location.	
XS_LN_TYP	Text	24		Cross-Section Line Type – Based on whether the cross section has a letter assigned in the XS_LTR field (LETTERED) or not (NOT LETTERED). Lettered cross sections are shown on the hardcopy FIRM.	LETTERED, NOT LETTERED
WTR_NM	Text	100		Surface Water Feature Name. This is the formal name of the stream or water body.	(Same value as WTR_NM in Stream Centerline/S_WTR_LN)
WSEL_REG	Double	Default		Modeled Water Surface Elevation for the 1-Percent-Annual-Chance Flood Event. This the modeled water-surface elevation for the 1-percent-annual-chance flood event in the stream channel at this cross section. In the case of levee(s) associated with a cross section, it is assumed that the levee(s) holds.	(Value from hydraulic model, rounded to two decimal places)
LEN_UNIT	Text	11		Water-Surface and Streambed Elevation Units. This unit indicates the measurement system used for the water-surface and streambed elevations.	CENTIMETERS, FEET, INCHES, KILOMETERS, METERS, MILES, MILES
V_DATUM	Text	17		Vertical Datum. The vertical datum indicates the reference surface from which the flood and streambed elevations are measured.	MSL, NAVD88, NGVD29, LOCAL TIDAL DATUM, MLLW, MLW
SOURCE_CIT	Text	11		(Leave field empty)	(Leave field empty)

Floodplain

Feature Name: S_FLD_HAZ_AR

Geometry Type: Polygon

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
FLD_AR_ID	Text	25		Unique feature identifier.	(Leave field empty)
FLD_ZONE	Text	55		Flood Zone - the flood zone designation used by FEMA to designate the SFHAs and for insurance rating purposes.	A, AE, AH, AO, AR, A99, B, C, D, V, VE, X, 0.2 PCT ANNUAL CHANCE FLOOD HAZARD, 1 PCT ANNUAL CHANCE FLOOD HAZARD CONTAINED IN CHANNEL, 1 PCT FUTURE CONDITIONS
FLOODWAY	Text	8		Floodway Area - populated if the area is within a regulatory floodway.	FLOODWAY, Null
SFHA_TF	Text	1		Special Flood Hazard Area. If the area is within a SFHA this field would be true. This field will be true for any area coded as an A or V flood zone area. It should be false for any X or D flood areas.	T, F, U (True, False, Unknown)
STATIC_BFE	Double	Default	2	Static Base Flood Elevation. This field will be populated for areas that have been determined to have a constant Base Flood Elevation (BFE) over a flood zone such as a lake or coastal zone where the BFE applies to the entire polygon. The BFE value will be shown beneath the zone label.	
V_DATUM	Text	17		Vertical Datum - indicates the reference surface from which the flood elevations are measured. This field is only populated if the STATIC_BFE field is populated.	MSL, NAVD88, NGVD29, LOCAL TIDAL DATUM, MLLW, MLW
DEPTH	Double	Default	2	Depth. This is the depth for Zone AO areas. This value is shown beneath the zone label on the FIRM. This field is only populated if a depth is shown on the FIRM.	
LEN_UNIT	Text	11		Length Units - indicates the measurement system used for the BFEs and/or depths. This field is only populated if the STATIC_BFE or DEPTH field is populated.	CENTIMETERS, FEET, INCHES, KILOMETERS, METERS, MILES
VELOCITY	Double	Default	2	Velocity - the velocity measurement of the flood flow in the area. Normally this is applicable to alluvial fan areas (certain Zone AO areas). This value is shown beneath the zone label on the FIRM. This field is only populated when a velocity is associated with the flood zone area.	
VEL_UNIT	Text	17		Velocity Unit - the unit of measurement for the velocity. This field is populated when the VELOCITY field is populated.	CENTIMETERS/HOUR, FEET/SECOND, INCHES/HOUR, METERS/SECOND

SOURCE_CIT	Text	11		(Leave field empty)	(Leave field empty)
Floodplain Feature Name: S_FLD_HAZ_LN Geometry Type: Polyline					
Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
DFIRM_ID	Text	6		Study Identifier. For a single-jurisdiction flood risk project, the value is composed of the two-digit State FIPS code and the four-digit FEMA CID code (e.g., 480001). For a countywide flood risk project, the value is composed of the two-digit State FIPS code, the three-digit county FIPS code, and the letter "C" (e.g., 48107C). Within each FIRM Database, the DFIRM_ID value will be identical.	(Same value as effective DFIRM.)
FLD_LN_ID	Text	25		Unique feature identifier	(Leave field empty)
LN_TYP	Text	45		Line Type. These line types describe the flood boundary and may be used to indicate how the feature must be depicted on the hardcopy FIRM. An example is provided in Figure 1 on page 5 of this appendix.	SFHA/FLOOD ZONE BOUNDARY, LIMIT OF DETAILED STUDY/LIMIT OF STUDY, OTHER BOUNDARY
SOURCE_CIT	Text	11		(Leave field empty)	(Leave field empty)

Junctions

Feature Name: S_HydroNode

Geometry Type: Points

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
NodeID	Text	25		Unique feature identifier	(Leave field empty)
NodeName	Text	254		Hydrologic node name and description of node location	
IsPourPt	Text	1		Sub-basin pour point (outlet). This field will be true for any point that represents a sub-basin outlet. It should be false for other points, which represent confluences.	T, F, U (True, False, Unknown)
SOURCE_CIT	Text	11		(Leave field empty)	(Leave field empty)

Land Use

Feature Name: LandUse

Geometry Type: Polygon

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
SARBCODE	Text	2		Land Use Code	See Table 10 in Section 3.4.2.
AREA	Double	Default		Area of land use polygons in acres.	

Reaches

Feature Name: S_HydroLink

Geometry Type: Points

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
LinkID	Long			Unique feature identifier	(Leave field empty)
WTR_NM_LID	Long			Surface Water Feature Name. This is the name of the surface water feature represented by the hydrologic reach link.	(Same value as WTR_NM in Stream Centerline/S_WTR_LN if corresponding to a mapped stream.)
LinkName	Text	254		Secondary feature identifier or group name. Populate field or (Leave field empty).	
UpNodeID	Text	25		Upstream Node. This is the node ID at the upstream end of the reach.	
DownNodeID	Text	25		Downstream Node. This is the node ID at the downstream end of the reach.	
SOURCE_CIT	Text	11		(Leave field empty.)	(Leave field empty)

Stream Centerline

Feature Name: S_WTR_LN

Geometry Type: Polyline

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
WTR_LN_ID	Text	25		Unique feature identifier	(Leave field empty)
WATER_TYP	Text	25		Surface Water Feature Type. The type value describes the kind of watercourse represented.	AREA OF COMPLEX CHANNELS/OVERFLOWS, STREAM CENTERLINE, OPEN WATER AREA, WETLANDS, MANMADE WATER FEATURE, GLACIAL FEATURE, COASTLINE/ISLAND SHORELINE, INTERMITTENT RIVER/STREAM/WASH
CHAN_REP	Text	6		Channel Representation. Single means linear water features represented by a centerline. Double means linear water features represented by shorelines or channel banks.	SINGLE, DOUBLE
WTR_NM	Text	100		Surface Water Feature Name. This is the formal name of the surface water feature, as it will appear on the hardcopy FIRM.	
SOURCE_CIT	Text	11		(Leave field empty)	(Leave field empty)

Structures

Feature Name: S_GEN_STRUCT

Geometry Type: Points

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
STRUCT_ID	Text	25		Unique feature identifier.	(Leave field empty)
STRUCT_TYP	Text	25		Structure Type. These are hydraulic structures within the flood risk project area.	BRIDGE, CANAL, CHANNEL, CULVERT, DAM, DIKE, DROP STRUCTURE, ENERGY DISSIPATER, FLUME, GATE, LEVEE, PUMP STATION, SIDE WEIR STRUCTURE, STORM SEWER, UTILITY CROSSING, WEIR, WING WALL, FLOODWALL, PIPELINE, RETAINING WALL, SIPHON, OTHER/MISC STRUCTURE
STRUCT_NM	Text	50		Structure Name. This is the proper name of the feature and/or the related transportation feature name as shown on the FIRM and/or the flood profile. If the flood profile has the proper structure name and no related transportation name, this field stores the proper name (e.g., Hoover Dam). If the flood profile has the related transportation name and no proper name, this field stores the related transportation name (e.g., Main Street). If the flood profile has the proper name and the transportation name, this field stores both names (e.g., Hoover Dam / Main Street). If structure has no proper name and no related transportation name, this field is left blank; this field should not store the structure type (e.g., dam).	
WTR_NM	Text	100		Surface Water Feature Name. This is the formal name of the surface-water feature associated with the structure, as it will appear on the hardcopy FIRM.	(Same value as WTR_NM in Stream Centerline/S_WTR_LN)
SOURCE_CIT	Text	11		(Leave field empty.)	(Leave field empty)

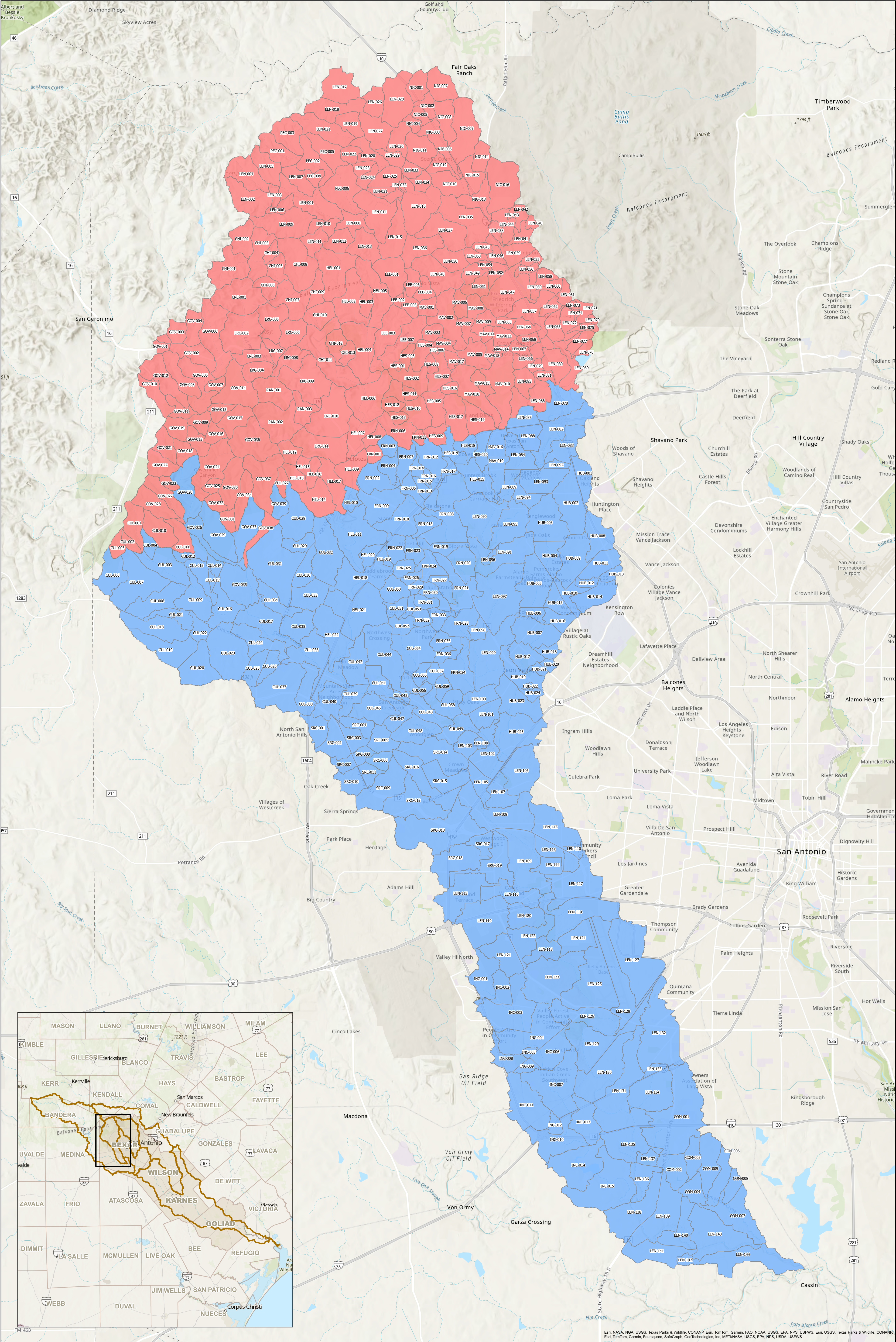
Sub-basins

Feature Name: S_HydroBasin

Geometry Type: Polygon

Field Name	Type	Length/ Precision	Scale	Field Description	Values
OBJECTID	Double	11		An automatically generated unique identifier.	
BasinID	Text	25		Unique feature identifier.	(Leave field empty)
BasinName	Text	254		Name of sub-basin.	
WTR_NM_LID	Text	100		Surface Water Feature Name. This is the name of the primary flooding source drained by the sub-basin.	
NodeID	Text	25		Node Identification. The node is associated with the sub-basin.	
Shape_Area	Double	Default		Area of sub-basin in acres.	
SOURCE_CIT	Text	11		(Leave field empty.)	(Leave field empty)

Appendix F: Snyder Peaking Coefficient Maps



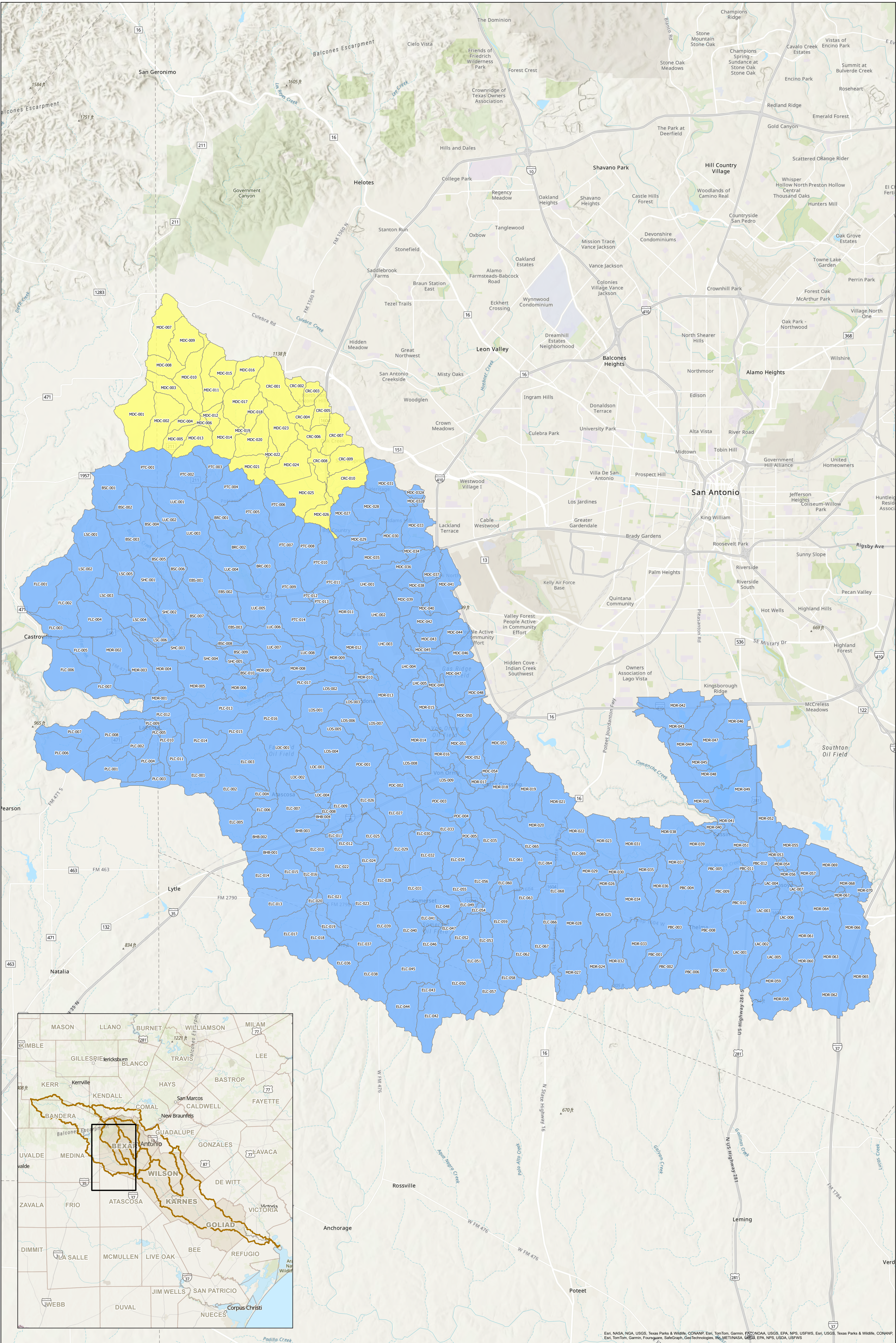
Snyder Peaking Coefficient

- Flat Slope (Cp = 0.55)
- Moderate Slope (Cp = 0.61)
- Steep Slope (Cp = 0.71)

Snyder Peaking Coefficients Map

Leon Creek

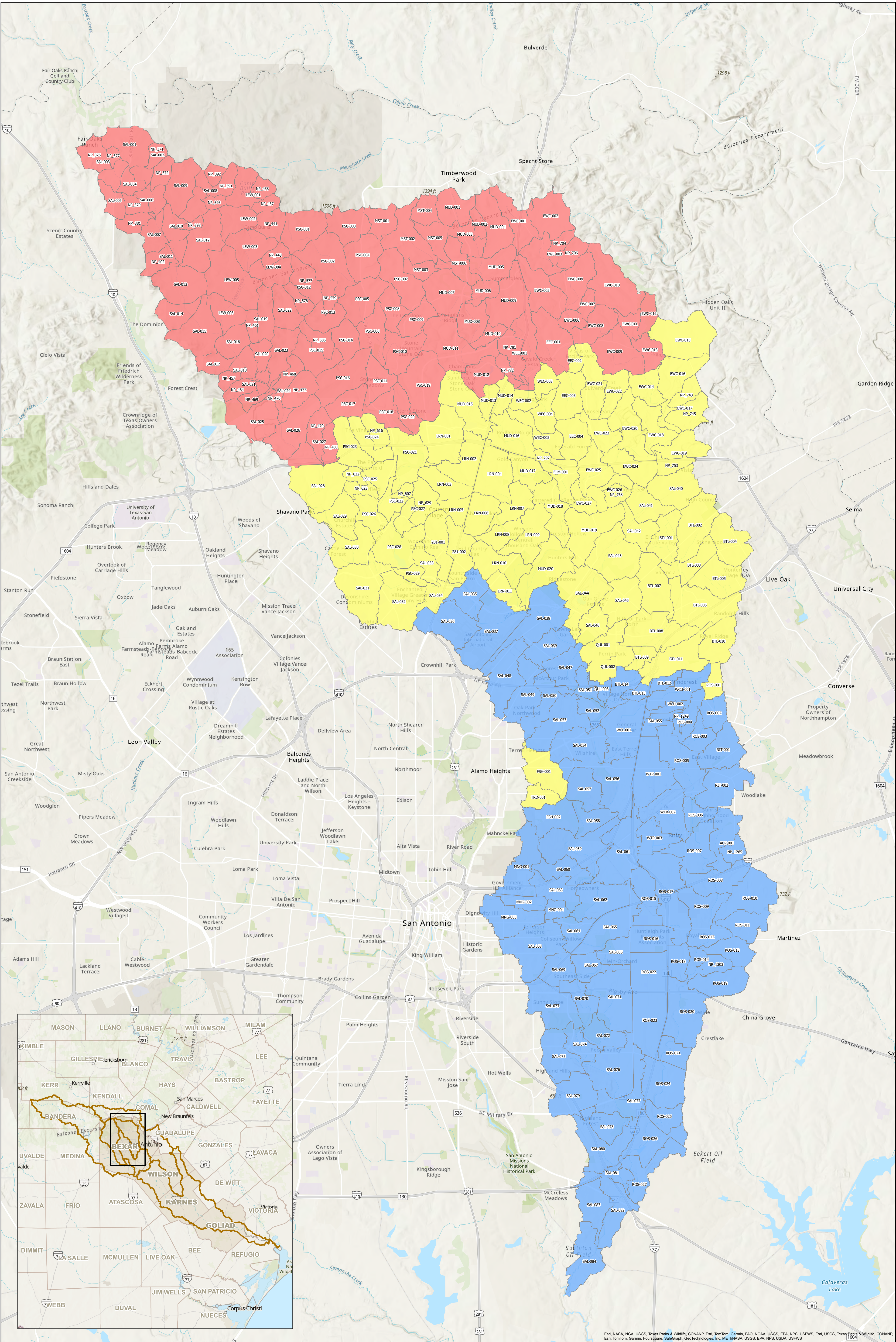
March 2025



- Snyder Peaking Coefficient**
- Flat Slope (Cp = 0.55)
 - Moderate Slope (Cp = 0.61)
 - Steep Slope (Cp = 0.71)

Snyder Peaking Coefficients Map
Lower Medina River

March 2025



Esri, NASA, NGA, USGS, Texas Parks & Wildlife, CONANP, Esri, TomTom, Garmin, FAO, NOAA, USGS, EPA, NPS, USFWS, Esri, USGS, Texas Parks & Wildlife, CONANP, Esri, TomTom, Garmin, FourSquare, SafeGraph, GeoTechnologies, Inc., METI/NASA, USGS, EPA, NPS, USDA, USFWS



0124

Miles

N

Snyder Peaking Coefficient

Flat Slope (Cp = 0.55)

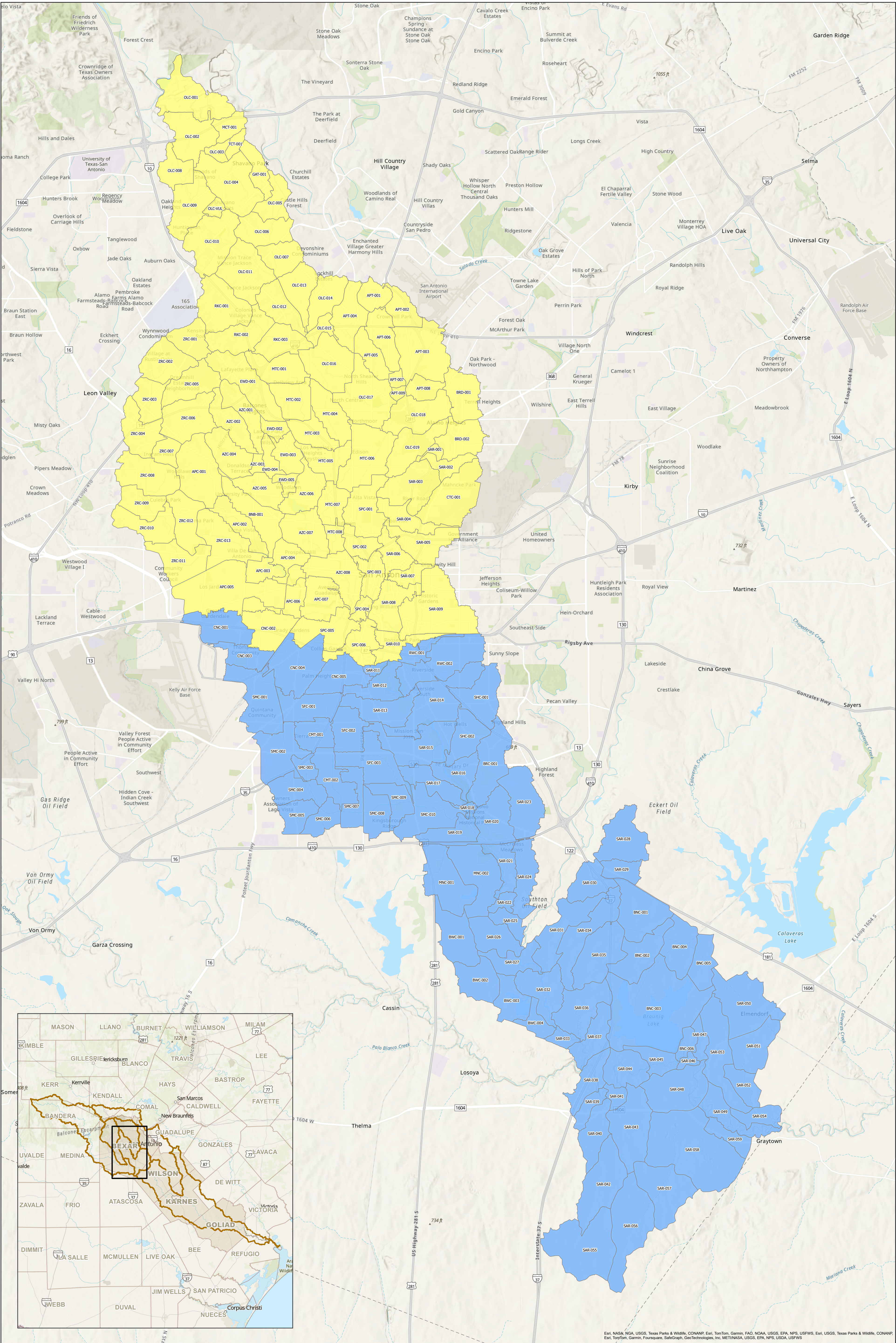
Moderate Slope (Cp = 0.61)

Steep Slope (Cp = 0.71)

Snyder Peaking Coefficients Map

Salado Creek

March 2025



Esri, NASA, NGA, USGS, Texas Parks & Wildlife, CONANP, Esri, TomTom, Garmin, FAO, NOAA, USGS, EPA, NPS, USFWS, Esri, USGS, Texas Parks & Wildlife, CONANP, Esri, TomTom, Garmin, Fourquare, SafeGraph, GeoTechnologies, Inc, METINASA, USGS, EPA, NPS, USDA, USFWS