MHFD
MILE HIGH FLOOD DISTRICT
12575 W Bayaud Ave | Lakewood, Colorado 80228
Phone. 303.455.6277 | www.mhfd.org

# TECHNICAL MEMORANDUM

| | |
|---|---|
| Date: | August 24, 2022 (Revised 1/11/2024) |
| Analysis by: | Brik Zivkovich, P.E., CFM |
| Review by: | Holly Piza, P.E., D. WRE |
| Re: | Investigation of the Water Quality Event and Recommendations for Calculating Water Quality Peak Flows |

## INTRODUCTION

The purpose of this memorandum is to document the investigation of the water quality event (WQE) and evaluate methods for determining the water quality peak flow (WQPF). For this memorandum, Mile High Flood District (MHFD) defines the WQE as a design storm representing a rainfall depth equal to the 80th percentile runoff-producing storm event for the Denver metropolitan region. The design storm depth corresponding to the water quality event (WQE) is 0.60 inches (0.60") for the Denver metropolitan region. This regional design storm depth is used to estimate a water quality capture volume (WQCV). The WQCV, initially developed in 1989 and revised in 1996, is the basis for determining the desired volume capture rate (between 80-90%) of the stormwater facility (Urbonas et al., 1990; Guo & Urbonas, 1996). The WQCV Method is internationally accepted and intends to target stormwater pollutants in urban runoff. Therefore, MHFD recommends using stormwater control measures (SCMs) that provide the WQCV with a slow release (USDCM, 2010). However, the method does not produce the information needed to design specific SCM components (or flow-based treatment practices). For example, a hydrodynamic separator (HDS) – a type of SCM and sedimentation-based manufactured treatment device (MTD) – has a removal efficiency that is dependent upon device characteristics, stormwater discharge, water temperature, and influent properties (sediment characteristics) from which volume-based design methodologies cannot be applied in suitably for those types of designs (Wilson et al. 2008).

Subsequent sections of the memorandum intend to close that design gap and support volume-based and flow-based stormwater designs. A two-part technical analysis and recommended regional design metrics and methods are presented for determining WQE discharges. The procedures presented in this document support recommended computational methods of designing storm runoff that align with the regionally accepted approaches presented in the latest editions of the Urban Storm Drainage Criteria Manual (USDCM). Final recommendations from this analysis are discussed and used as a basis of design to support regional criteria updates for the USDCM.

## METHODS

The WQE investigation in this technical memorandum considered two different approaches to investigate the WQE and evaluate WQE discharges. These approaches include (1) rainfall-only assessment and (2) rainfall-runoff assessments. The first analysis (rainfall only) investigated regional storm event characteristics, which were assessed without considering runoff data or calculations. The rainfall-only analysis looks at correlations of storm event characteristics. (storm event depths, intensities, duration, return periods, distribution, and seasonality). The second analysis investigated rainfall-runoff relationships of smaller storm events through a comparison of two regionally accepted runoff methods commonly used for stormwater designs. The two methods, the Rational Method and the Colorado Urban Hydrograph Procedure (CUHP), and the corresponding design inputs are evaluated and compared to further investigate peak flow estimates during the water quality storm event and provide recommended guidance on the WQPF for the USDCM.

### Part 1: Rainfall Analysis

The detailed rainfall analysis investigated the characteristics of smaller regional storm events. This analysis, which expands on previous research, was completed as the first phase of this memorandum with support from Dr. James CY Guo. The rainfall analysis is further discussed in the memorandum by Dr. Guo (Guo 2021) and summarized in detail through a research roadmap in Appendix A. Key findings and observations from these analyses are presented and further discussed in Results & Discussion.

### Part 2: Rainfall-Runoff Analysis

The second part of this memorandum performs a multi-variate analysis of the WQE to evaluate and compare rainfall-runoff estimates between two regionally accepted methods (Rational Method & CUHP) used for determining the WQPF. The first design method investigated is a Rational Method to find the peak flows for a given return period. The second method investigated is a regionally calibrated hydrologic routing approach known as the Colorado Urban Hydrograph Procedure (CUHP).

Two key assumptions are incorporated into the rainfall-runoff analysis to compare WQE discharges determined from either method. The first assumption is that the WQE precipitation depth equals 0.60” (undefined distribution). The second assumption is that the design parameters for storm depth and drainage area remain dependent variables in both methods, meaning these variables are constant when applied to produce a given set of results to compare WQPF between both methods.

## WATER QUALITY PEAK FLOW (via Rational Method)

The following section summarizes calculations of WQPF using the Rational Method. Theoretically, this method can be used to estimate a peak flow rate based on a minimal set of design parameters. The design parameters used in this analysis are based on (1) runoff coefficients derived from imperviousness by hydrologic soil group and storm duration (based on USDCM Table 6-4), (2) average rainfall intensity with a one-hour point precipitation depth (P1=0.60”) using storm duration equal to the time of concentration (based on USDCM Equation 5-1), and (3) drainage area. The design inputs used in the Rational Method analysis are summarized in Table 1. The calculated storm intensities for the WQE analysis via the Rational Method are in Table 2.

***Table 1. Summary of Input Parameters for WQE Analysis using Rational Method***

| Parameter | Input Values |
|---|---|
| Drainage area, A (ac) | 1 ac |
| Precipitation depth, P1 (in) | 0.60” |
| Design intensity, I (in/hr) | Based on “I” calculated via USDCM Equation 5-1<br>Based on “I” calculated via Guo & Urbonas (2021) |
| Imperviousness, I (%) | 100%, 80%, 60%, 40%, 20% |
| Runoff coefficient, C (-) | Calculated based on %I per USDCM Table 6-4 (with 2-year and soil type C/D) |
| Storm duration, Td (min) | 5-, 10-, 15-, 20-, 30-, 45-, 60-min |

***Table 2. Water Quality Event Design Intensities based on Regional Intensity Equations***

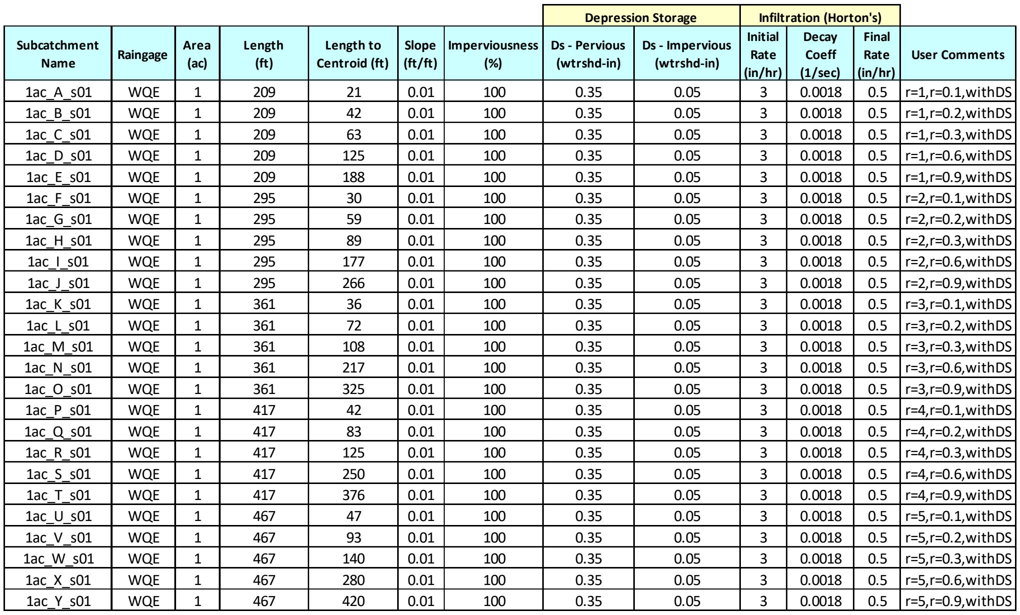
| Return Period | P1 (in) | Method | i (in/hr) $T_d = 5$ min | i (in/hr) $T_d = 10$ min | i (in/hr) $T_d = 15$ min | i (in/hr) $T_d = 20$ min | i (in/hr) $T_d = 30$ min | i (in/hr) $T_d = 60$ min |
|---|---|---|---|---|---|---|---|---|
| WQE | 0.60 | USDCM Equation 5-1 | 2.04 | 1.62 | 1.36 | 1.18 | 0.94 | 0.61 |

## WATER QUALITY PEAK FLOW (via Colorado Urban Hydrograph Procedure)

The following section discusses the WQPF calculation based on a Colorado Urban Hydrograph Procedure (CUHP). CUHP can be used develop a storm hydrograph based corresponding to a rainfall design storm distribution and watershed characteristics. Unlike the Rational Method, a point precipitation depth is used to distribute precipitation and create a design hyetograph, then a storm hydrograph is calculated used a set of watershed characteristics (area, imperviousness, length, length-to-centroid, slope, depression storage, and infiltration). The rainfall distribution approach in CUHP (point-precipitation depth distributed over a two hour period), which was investigated in the rainfall analysis of this memorandum (Appendix A), is shown to be comparable to rainfall records (Guo, 2021), meaning the 2-year distribution will be applied to investigate the WQE using CUHP. The design parameters used in this analysis assume (1) the WQE precipitation depth of 0.60 inches (0.60") is appropriate for P1, (2) the 2-year, 2-hour design storm distribution is representative and appropriate for the WQE rainfall distribution (Guo, 2021), and (3) CUHP is suitable for calculating peak runoff based on the subbasin characteristics at hydrologic design scale (less than 5 acres). Table 3 and Table 4 summarize the design inputs of the different parameters used in the CUHP model scenarios. Design inputs for the results of various CUHP subbasin configurations and scenario runs are presented in Appendix C.

***Table 3. Summary of Input Parameters for WQE Analysis using CUHP***

| CUHP Input Parameters | Input Values |
|---|---|
| Drainage Area, A (ac) | 1 ac. |
| Precipitation depth, P1 (in) | 0.60” |
| Imperviousness, I (%) | 100%, 80%, 60%, 40%, 20% |
| Length, L (ft) | By R-values<br>R=[1, 2, 3, 4, 5] |
| Length-to-centroid, Lc (ft) | By length and r-value<br>R=[0.1, 0.2, 0.3, 0.6, 0.9] |
| Slope, S (ft/ft) | 0.01, 0.03, 0.05 |
| Depression Losses, Ds (in) | Impervious - 0.05"<br>Pervious - 0.35" |
| Infiltration Rates | Soil Type C/D<br>Initial rate = 3 in/hr<br>Decay Coefficient = 0.0018<br>Final rate = 0.5 in/hr |
| Storm event distribution* | CUHP, 2-year, 2-hour |
| CUHP Computation Time Step | 1-minute |



| | | | | | | | Depression Storage | | Infiltration (Horton's) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Subcatchment Name | Raingage | Area (ac) | Length (ft) | Length to Centroid (ft) | Slope (ft/ft) | Imperviousness (%) | Ds - Pervious (wtrshd-in) | Ds - Impervious (wtrshd-in) | Initial Rate (in/hr) | Decay Coeff (1/sec) | Final Rate (in/hr) | User Comments |
| 1ac_A_s01 | WQE | 1 | 209 | 21 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=1,r=0.1,withDS |
| 1ac_B_s01 | WQE | 1 | 209 | 42 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=1,r=0.2,withDS |
| 1ac_C_s01 | WQE | 1 | 209 | 63 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=1,r=0.3,withDS |
| 1ac_D_s01 | WQE | 1 | 209 | 125 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=1,r=0.6,withDS |
| 1ac_E_s01 | WQE | 1 | 209 | 188 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=1,r=0.9,withDS |
| 1ac_F_s01 | WQE | 1 | 295 | 30 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=2,r=0.1,withDS |
| 1ac_G_s01 | WQE | 1 | 295 | 59 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=2,r=0.2,withDS |
| 1ac_H_s01 | WQE | 1 | 295 | 89 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=2,r=0.3,withDS |
| 1ac_I_s01 | WQE | 1 | 295 | 177 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=2,r=0.6,withDS |
| 1ac_J_s01 | WQE | 1 | 295 | 266 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=2,r=0.9,withDS |
| 1ac_K_s01 | WQE | 1 | 361 | 36 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=3,r=0.1,withDS |
| 1ac_L_s01 | WQE | 1 | 361 | 72 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=3,r=0.2,withDS |
| 1ac_M_s01 | WQE | 1 | 361 | 108 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=3,r=0.3,withDS |
| 1ac_N_s01 | WQE | 1 | 361 | 217 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=3,r=0.6,withDS |
| 1ac_O_s01 | WQE | 1 | 361 | 325 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=3,r=0.9,withDS |
| 1ac_P_s01 | WQE | 1 | 417 | 42 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=4,r=0.1,withDS |
| 1ac_Q_s01 | WQE | 1 | 417 | 83 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=4,r=0.2,withDS |
| 1ac_R_s01 | WQE | 1 | 417 | 125 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=4,r=0.3,withDS |
| 1ac_S_s01 | WQE | 1 | 417 | 250 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=4,r=0.6,withDS |
| 1ac_T_s01 | WQE | 1 | 417 | 376 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=4,r=0.9,withDS |
| 1ac_U_s01 | WQE | 1 | 467 | 47 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=5,r=0.1,withDS |
| 1ac_V_s01 | WQE | 1 | 467 | 93 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=5,r=0.2,withDS |
| 1ac_W_s01 | WQE | 1 | 467 | 140 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=5,r=0.3,withDS |
| 1ac_X_s01 | WQE | 1 | 467 | 280 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=5,r=0.6,withDS |
| 1ac_Y_s01 | WQE | 1 | 467 | 420 | 0.01 | 100 | 0.35 | 0.05 | 3 | 0.0018 | 0.5 | r=5,r=0.9,withDS |

***Figure 1. CUHP Model Inputs Scenarios (Excl. Slopes = 0.03 ft/ft and 0.05 ft/ft)***

## RESULTS

The results of this memorandum are presented in two parts. The first part discusses the rainfall-only analysis of regional storm event characteristic assessments. The second part discusses the rainfall-runoff analysis related to comparing two methods (Rational Method and CUHP) used to determine WQPF.

### Part 1: Regional Storm Characteristics Assessment

Expanding on the previous literature and supporting work completed by Guo (2021), the key findings from the regional storm characteristics assessment include:

1) Rainfall intensity cannot be directly determined from rainfall depths as there was no direct correlation between storm event depths and storm intensity for the analyzed design storms (see Figure 2).
2) Rainfall intensity cannot be directly determined from storm duration as there was no direct correlation between storm duration and intensity for analyzed storm events (see Figure 3).
3) The normalization of the storm event to the regional design storm distribution indicates storm event trends follow the regional design storm distribution. Design storm distribution, which is used to develop the rainfall hyetograph in CUHP based on a one-hour point precipitation depth and corresponding return period, is valid for most small storm events regardless of other factors that may impact the statistical analysis (see Figure 4).
4) The time of separation between storm events and minimum storm event depth are the important variables that can significantly influence statistical analysis. These two variables affect how a storm event is normalized and define what is and what is not considered a qualifying storm event. These are also important factors when assessing hourly and sub-hourly datasets and when evaluating smaller, more frequent storm events with respect to the design storm distributions and rainfall intensity. Additional discussion is provided in Appendix A.

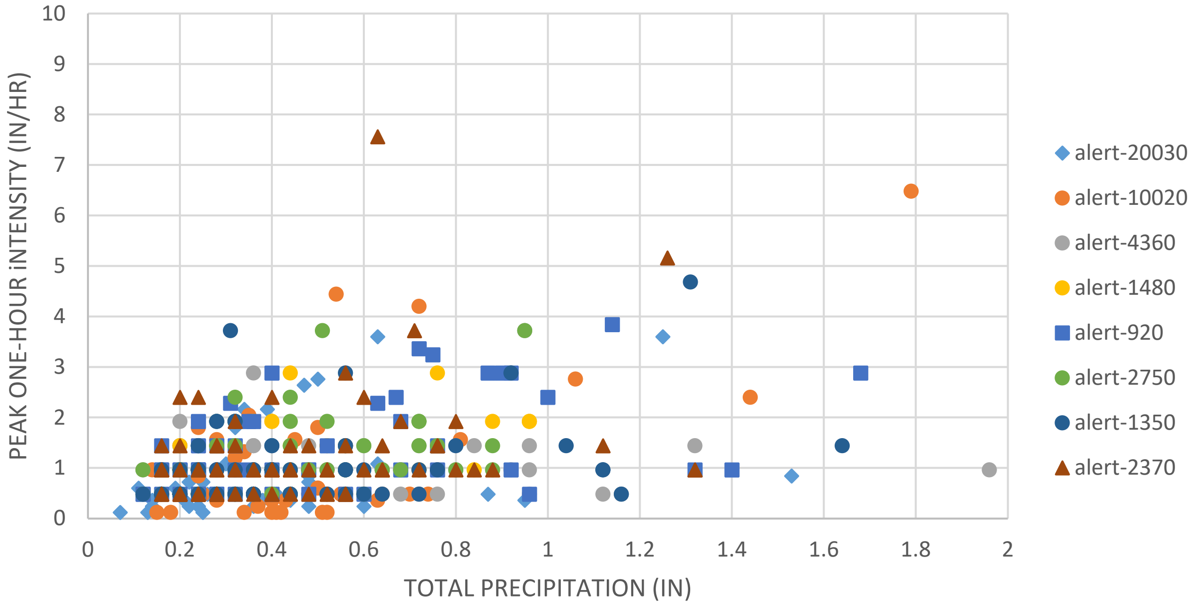


*Figure 2. Regional Storm Characteristics (Peak One-Hour Intensity vs. Total Precipitation)*

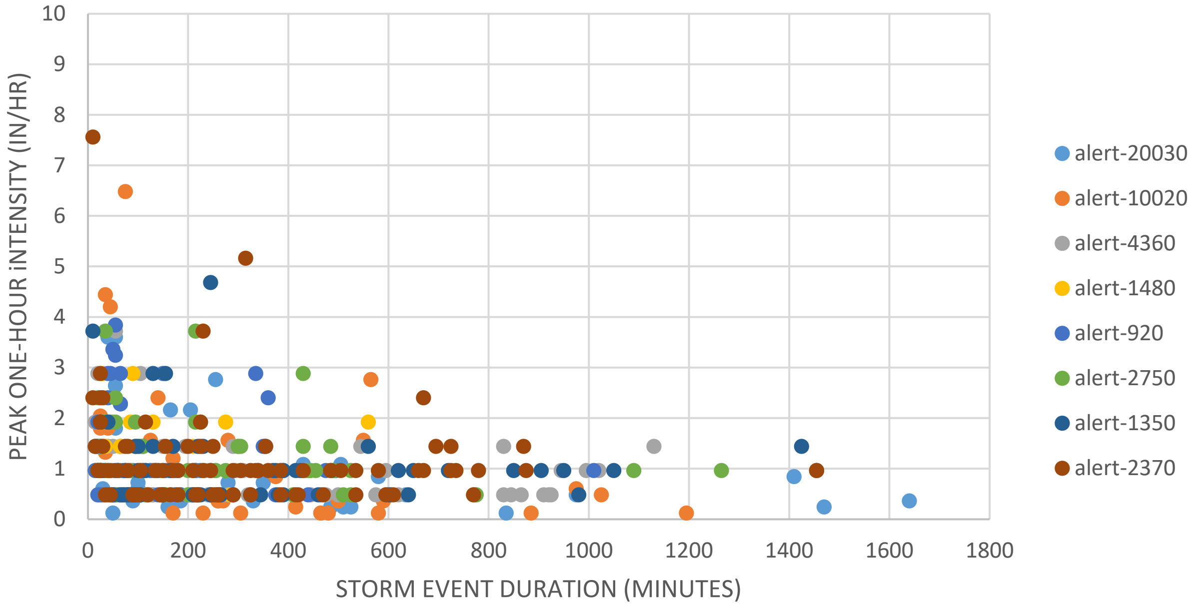


*Figure 3. Regional Storm Characteristics (Peak One-Hour Intensity vs. Storm Event Duration)*

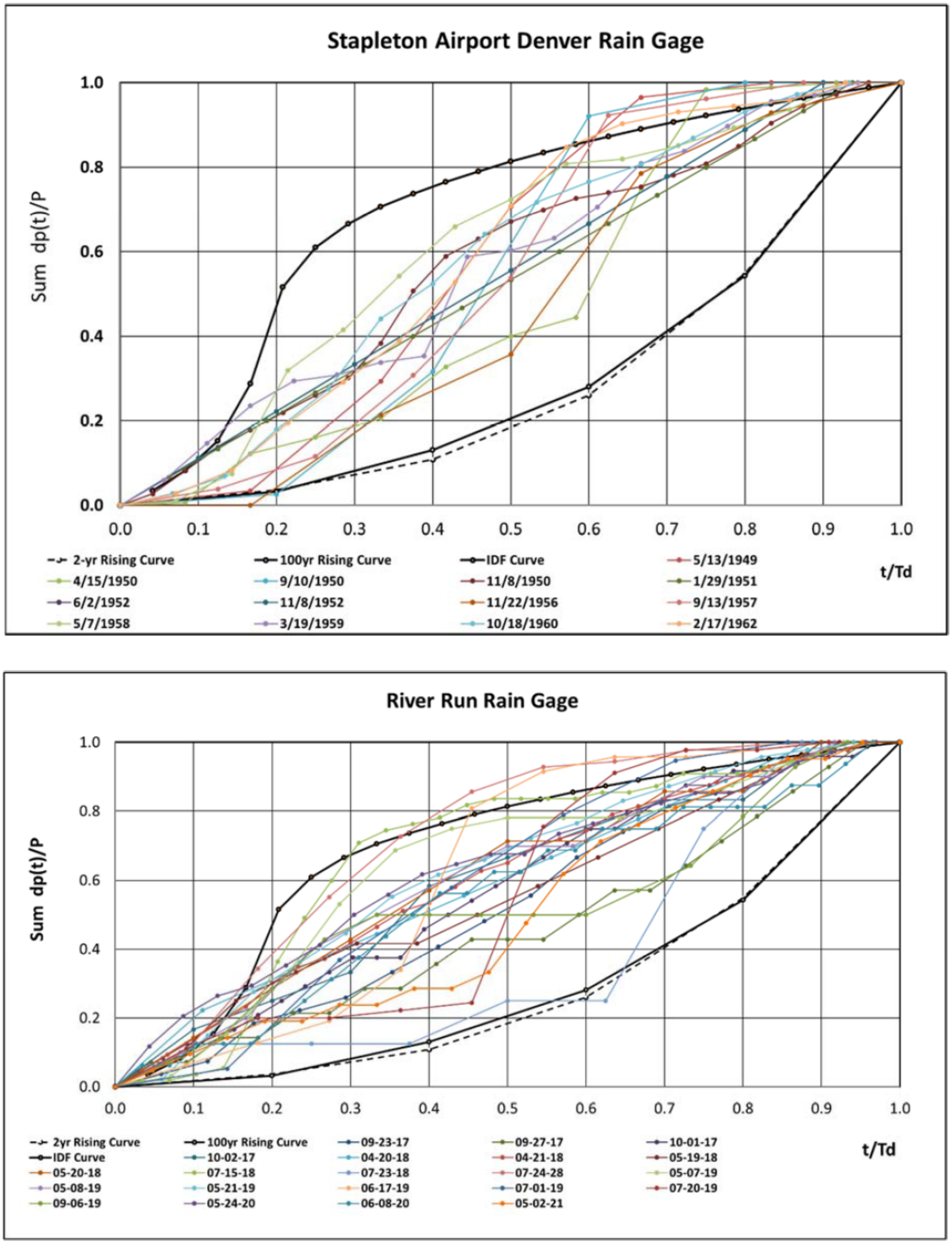


*Figure 4. Normalized Rainfall Events Recorded at Different Locations. [Reprint from Guo 2021]*

Part 2: Rainfall-Runoff Assessment

The results from the rainfall-runoff analysis to evaluate and compare WQPF estimates from the Rational Method and CUHP are summarized through tables and figures. Table 4 presents the rational method outputs. Table 5 shows the comparison of ranges for the two different methods. The range for the Rational Method is based on the minimum and maximum peak flow values, which correspond to the storm duration (Td = 5 min and Td=60 min). The range for the CUHP analysis is based on the minimum and maximum values, which correspond to the slope, length, and length-to-centroid relationships). Figures 5-9 plot both methods on a single graph to visually compare results from the Rational Method and CUHP results for an area of 1 acre. The Rational Method is represented as a straight line for illustration and comparison purposes only because only one peak flow value can be derived for a given storm event and storm duration, whereas all CUHP scenarios are plotted and represent the storm hydrographs. Additional discussion on the comparative analysis of rainfall-runoff methods for determining the WQE is presented in the Discussion & Recommendations section of this memorandum.

***Table 4. WQPF Calculations using Rational Method (P1=0.60" and A=1ac)***

| **Imperviousness, I (%) =** | | 100 | 80 | 60 | 40 | 20 |
|---|---|---|---|---|---|---|
| **Runoff Coefficient, C (-) =**<br>**Based on USDCM Table 6-4 equations (for WQE/2yr and Soil Type C/D)* | | 0.83 | 0.65 | 0.47 | 0.30 | 0.14 |
| **Td (min)** | **Avg. Rainfall Intensity, i (in/hr)**<br>**Based on USDCM Equation 5-1* | **Q (cfs) or WQPF (cfs)** | | | | |
| 5 | 2.04 | 1.69 | 1.32 | 0.95 | 0.60 | 0.28 |
| 10 | 1.62 | 1.35 | 1.05 | 0.76 | 0.48 | 0.22 |
| 15 | 1.36 | 1.13 | 0.88 | 0.64 | 0.40 | 0.19 |
| 20 | 1.18 | 0.98 | 0.76 | 0.55 | 0.35 | 0.16 |
| 30 | 0.94 | 0.78 | 0.61 | 0.44 | 0.28 | 0.13 |
| 45 | 0.73 | 0.61 | 0.47 | 0.34 | 0.22 | 0.10 |
| 60 | 0.61 | 0.50 | 0.39 | 0.28 | 0.18 | 0.08 |

***Table 5. Comparison of Water Quality Peak Flowrates (Rational Method vs. CUHP)***

| **Scenario** | **WQPF (cfs)**<br>*Rational Method* | **WQPF (cfs)**<br>*CUHP* |
|---|---|---|
| 1 ac, 100% | 0.50 – 1.69 | 0.55 - 1.52 |
| 1 ac, 80% | 0.39 – 1.32 | 0.39 – 1.13 |
| 1 ac, 60% | 0.28 – 0.95 | 0.24 – 0.79 |
| 1 ac, 40% | 0.18 – 0.60 | 0.11 – 0.42 |
| 1 ac, 20% | 0.08 – 0.28 | 0.03 – 0.17 |

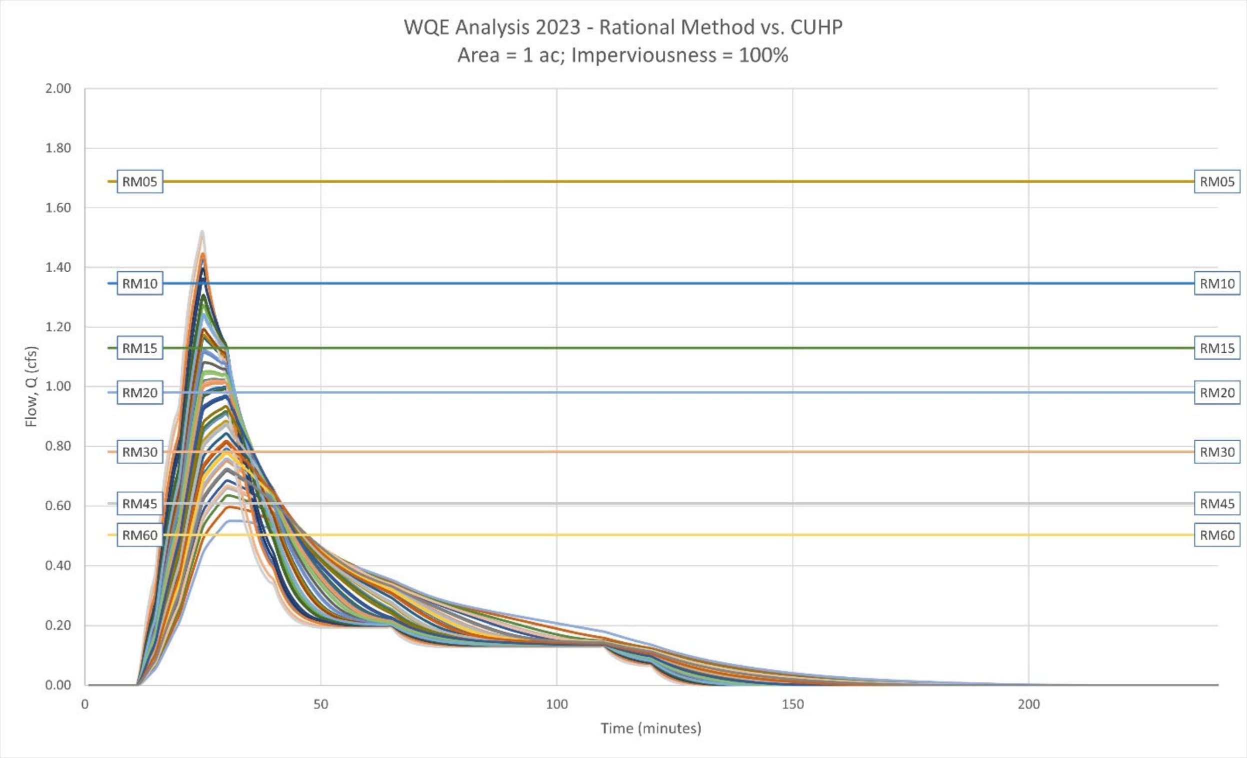


*Figure 5. Comparison of WQPF Results (1 acre with 100% imperviousness)*

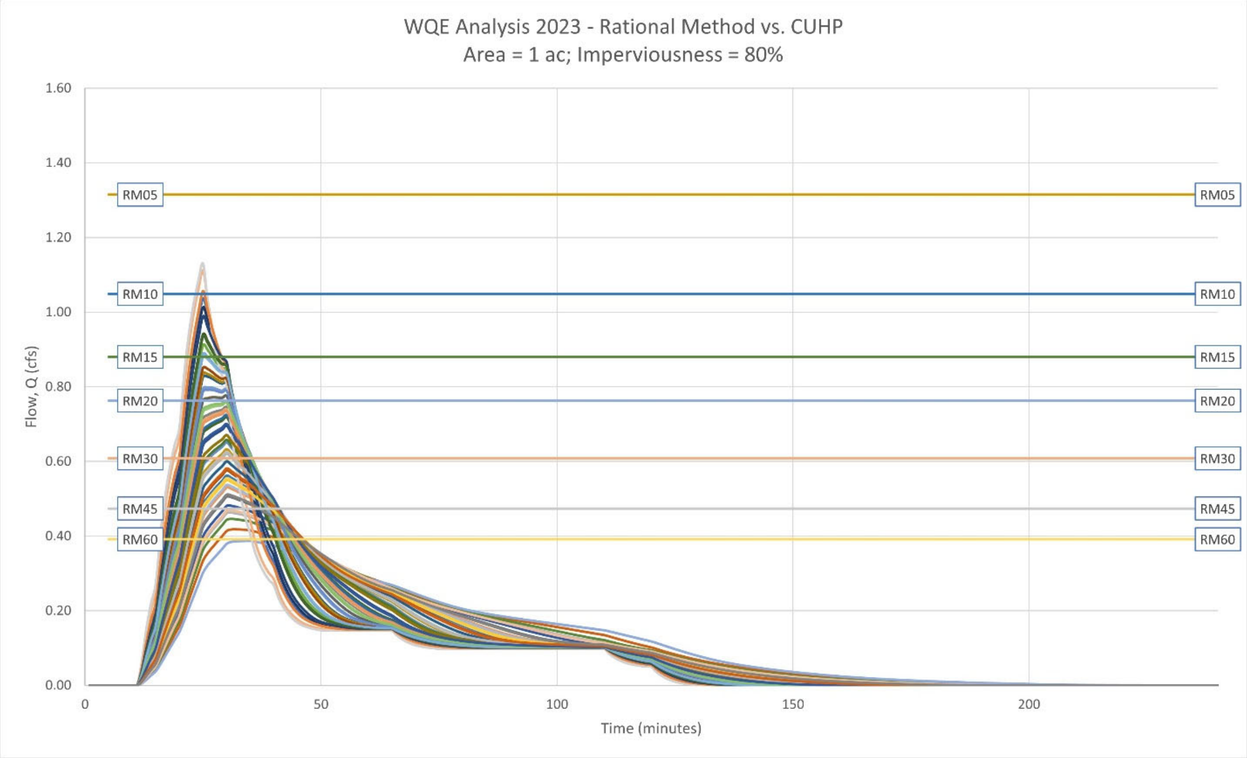


*Figure 6. Comparison of WQPF Results (1 acre with 80% imperviousness)*

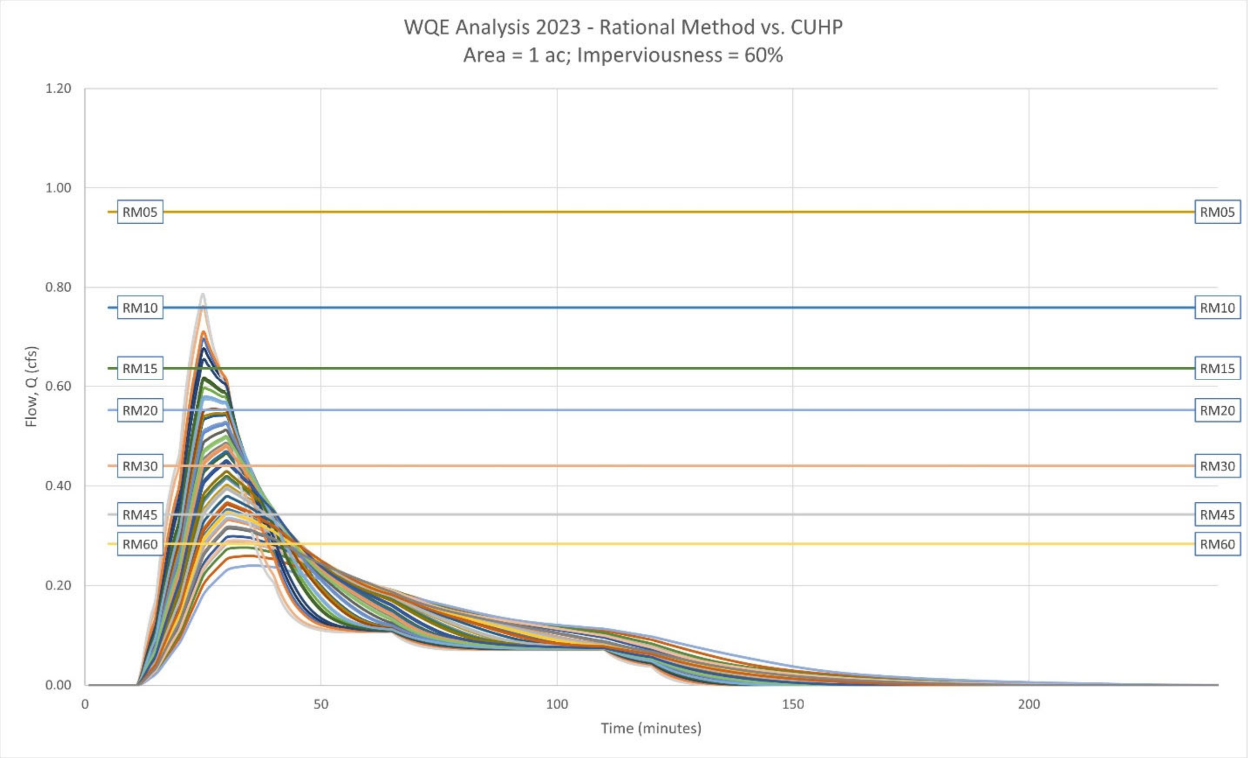


*Figure 7. Comparison of WQPF Results (1 acre with 60% imperviousness)*

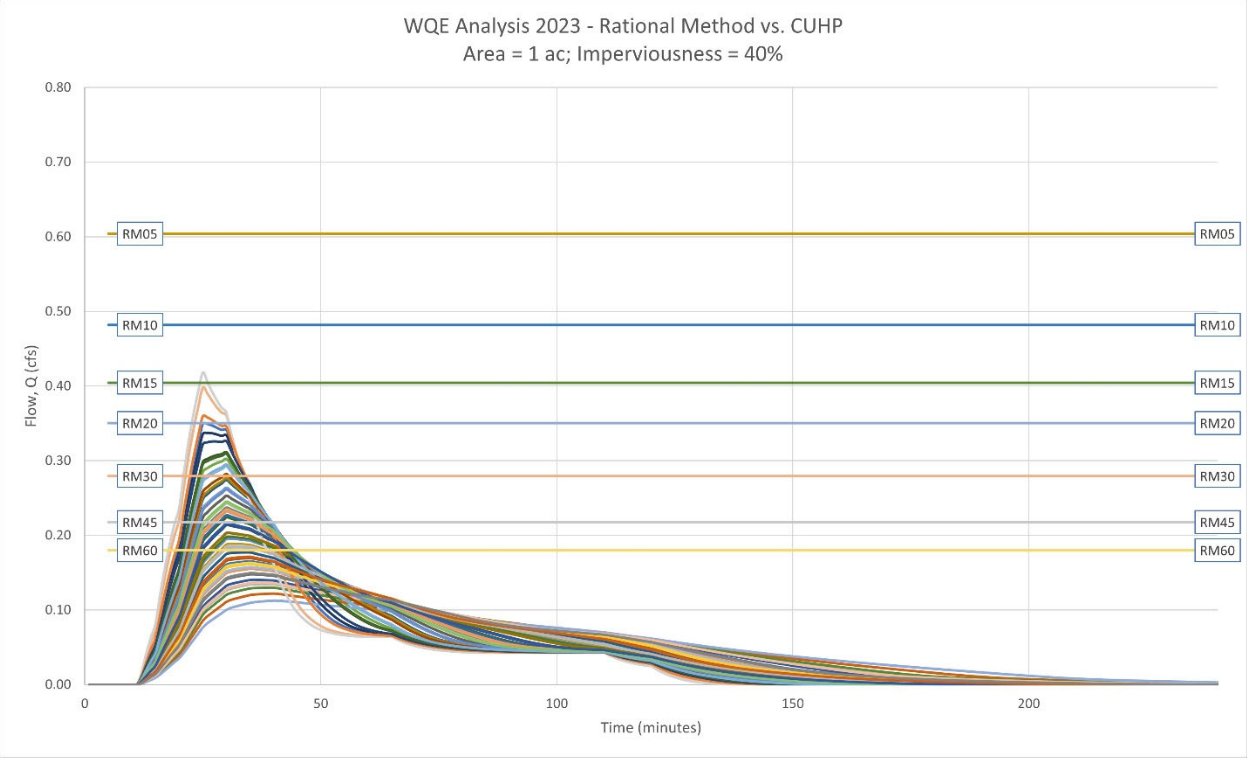


*Figure 8. Comparison of WQPF Results (1 acre with 40% imperviousness)*

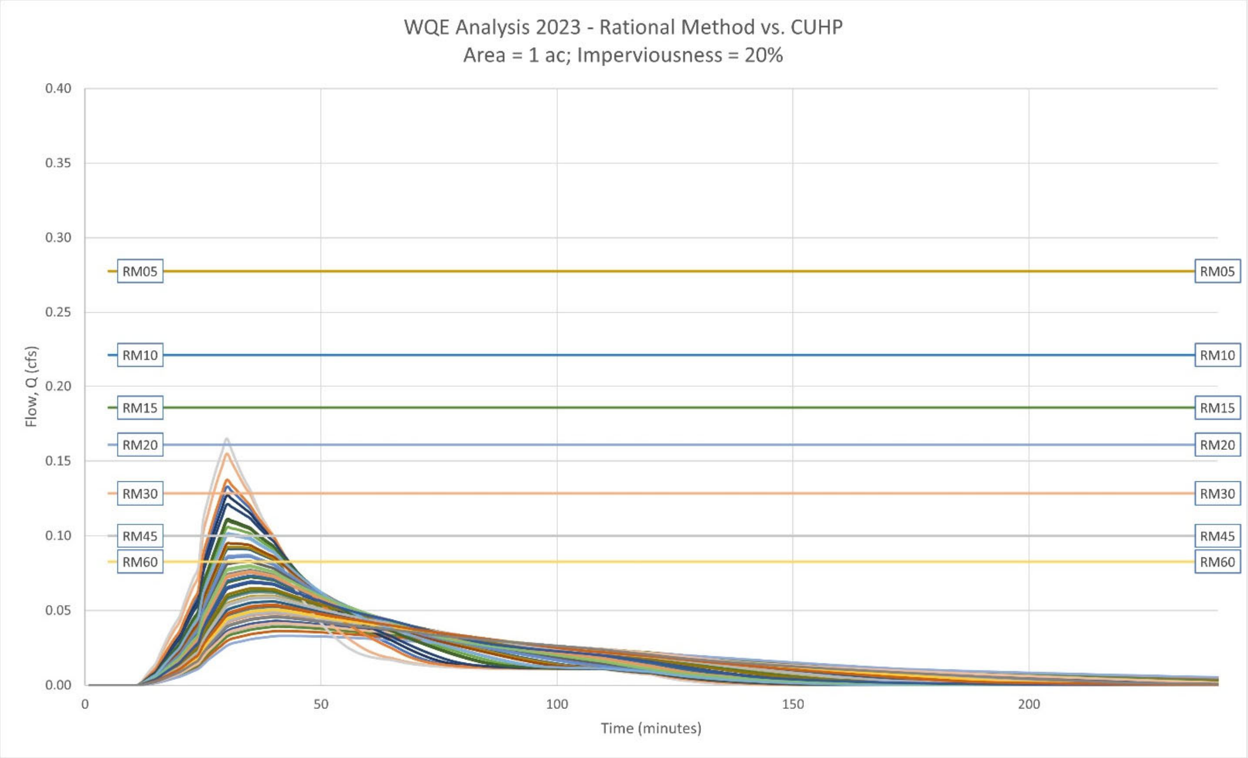


*Figure 9. Comparison of WQPF Results (1 acre with 20% imperviousness)*

## DISCUSSION

The WQE analysis suggests two primary drivers in differences between regionally accepted methods for determining peak discharges from the water quality event. These drivers refer to (1) storm event characteristics, and (2) design inputs and model parameterization used to represent drainage area characteristics at a given hydrologic scale.

1) Storm Event Characteristics

   For storm event characteristics, the rainfall analyses provided key findings about regional storm events and further illustrated where correlations do and do not exist. For example, regional storm events and rainfall intensity did not correlate directly with precipitation depth or duration. The key finding was that minor storm events (normalized to total depth and duration) commonly follow previously developed regional distribution patterns – short and intense storm events varying across all seasons.

2) Design Inputs and Model Parameterization

   Comparison of the runoff results between the two methods presents a direct challenge correlating results due to the inherent differences in design inputs and parameterization of each method. The Rational Method utilizes minimal input parameters (area, design rainfall intensity, and runoff coefficient) to develop one design parameter (peak flow). The peak discharge from the rational method is independent of the storm event hyetograph and designed storm hydrograph, and the design storm intensity is the average rainfall intensity for a duration equal to the time of concentration (Tc) and measured in inches/hour. In contrast, CUHP, which is a regionally calibrated unit hydrograph method, considers both (1) a rainfall distribution to produce a representative design storm hyetograph and (2) unique subbasin characteristics (length-to-centroid, length, imperviousness, and depression storage) to develop a representative storm hydrograph for a subcatchment. A quantitative comparison of the runoff results between the two methods (summarized in Table 5 and illustrated in Figures 5-9) indicate the range of values produces slightly lower peak flow rates when compared to the rational method. In this analysis, the Rational Method represents seven different model variations for a given imperviousness and area (variable intensity by Td – based on USDCM Equation 5-1) compared to the CUHP analysis producing 75 variations for a given area and storm event.

Further interpretation of the results and figures, it is apparent that for shorter duration storm events when Td is calculated to be less than 10-15 minutes, the rational method produces slightly higher peak flow values for the water quality event when compared to CUHP outlying scenarios. Additionally, as imperviousness decreases, peak flows based on the Rational Method are increasingly higher than those based on CUHP. It is important to note that the CUHP model shows extreme scenarios, variable by the R-values (i.e. using the recommended ranges of minimum and maximum allowed in CUHP – see CUHP manual for more information about the R-values for different input parameters). The key factors to have better alignment with the two methods are (a) adjustment factors on the CUHP timestep (modification from 5 minutes to 1 minute) for smaller storm events, including the WQE, and (b) adjustments to imperviousness used in the Rational Method, which are based on runoff coefficients corrections using USDCM Equation 6-4 with a 2-year return period and soil type C/D.

## RECOMMENDATIONS

Volume-based and flow-based approaches require different input parameters for engineering calculations. Currently, the USDCM presents volume-based methods for calculating runoff volume and sizing stormwater treatment facilities. Based on this analysis, which evaluated the storm event characteristics for smaller storm events and compared the estimated WQPF determined using the Rational Method and CUHP, either method is valid for calculating WQPF. There are known limitations to both methods; however, results from the assumptions considered in this analysis indicate some level of hydrologic agreement between the methods for evaluating WQPF. To maintain agreement in stormwater designs from various methods, there are additional considerations within each method to evaluate the WQE and derive a representative WQPF.

For the WQPF using the Rational Method, these considerations include:

1) Runoff coefficient adjustments using equations in USDCM Table 6-4 that are calculated from the imperviousness of the tributary area corresponding to a 2-year return period (WQE=2-year).
2) A representative soil group of the tributary area for calculating runoff from smaller, more frequent storm events.

For the WQPF via CUHP, these considerations include:

1) Using smaller timesteps in CUHP (1 minute, not 5 minutes)
2) A storm event distribution equivalent to the 2-year return period for the WQE
3) Design input parameters that align with CUHP recommended ranges for R-values that are used to represent drainage area characteristics (length, length-to-centroid, and slope).

Additionally, the selection of either method depends on the design application. If a storm hydrograph is required for stormwater design purposes (such as flow-based stormwater control measures or when routing multiple subcatchments through a drainage network), use CUHP to develop a storm hydrograph that can be routed through the storm drainage network or stormwater facility.

## REFERENCES

Guo, J. C., & Urbonas, B. (1996). Maximized detention volume determined by runoff capture ratio. *Journal of Water Resources Planning and Management*, *122*(1), 33-39.

Urbonas B., J. C. Y. Guo, and L. S. Tucker. (1989) updated 1990. Sizing Capture Volume for Stormwater Quality Enhancement. Flood Hazard News. Urban Drainage and Flood Control District: Denver, CO.

Wilson, M., J.S. Gulliver, O. Mohseni, and R.M. Hozalski. 2008. "Assessing the Effectiveness of Proprietary Stormwater Treatment Devices." *Journal of Water Management Modeling*. R228-14.

Urban Drainage and Flood Control District. (2010, November). *Urban Storm Criteria Drainage Manual, Volume 3.* Denver, CO

Urban Drainage and Flood Control District. (2018, August). *Urban Storm Drainage Criteria Manual, Volume 1.* Denver, CO.

Mile High Flood District (2019, October). "Chapter 3 - Calculating the WQCV and Volume Reduction" in *Urban Storm Drainage Criteria Manual Volume 3*. Mile High Flood District. Denver, CO.

Guo, J. (2021, August 25). *Derivation and Application of Water Quality Peak Flow for Metro Denver Area.* Technical Memorandum. Mile High Flood District. Denver, CO.

## APPENDICES

| | |
|---|---|
| Appendix A | Assessment of Regional Storm Event Characteristics |

## OVERVIEW

In 2021, MHFD contracted with James Guo to support rainfall data analyses in the MHFD region and develop a design storm distribution for the WQE. His work is summarized in a technical memorandum titled *Derivation and Application of Water Quality Peak Flow for Metro Denver Area*. (Guo, 2021). Appendix A presents a research roadmap documenting the background and approach used for the WQE analysis in order to define a recommended rainfall intensity for use in regional stormwater design equations. This research roadmap lays out the procedural techniques to obtain, review, and investigate regional storm event characteristics, and provides preliminary and supplemental background used for comparing the two methods presented for developing design method recommendations associated with the WQE. Supporting tables and figures are provided from this preliminary investigation for reference.

### WQE Research Roadmap

Step 1) Performed literature review of approved design storm methodologies for determining peak flows associated with water quality events (national/international).

Step 2) Reviewed original methods and dataset to develop the WQCV method (Stapleton, CO).

Step 3) Evaluated inputs, models, and outputs and determined limitations of dataset to develop a design storm intensity for the WQE. Limitations were defined as a function of the original source data, minimum precipitation storm depth, and duration between storm events (referred to as data noise or model noise). Data noise includes the data sets, corrected versions of the data sets, and data reporting such as intervals and procedures. Model noise includes the methods and assumptions of those methods such as minimum rainfall depth and separation time between qualifying storm events.

Step 4) Expanded original WQCV method to investigate sensitivity of outputs with respect to data inputs and models using expanded WQE analysis algorithms developed through information theory and set theory. The expanded algorithm can be viewed as a universal method for developing design storm intensity for the water quality event.

Step 5) WQCV Method – revisited and expanded original WQCV method to evaluate and assess sensitivity of the outputs (distribution of storm event depths based on one-hour precipitation totals) with respect to different inputs (different geographic and topographic locations) and models/model settings (duration between storm events, minimum storm depths). Original analysis included one rain gage with defined model settings for qualified storm events – $P_{min}$ (minimum precipitation depth) of at least 0.1 inches; Ts (time of separation between measurable precipitation) of at least 11.5 hours. This analysis expanded on previous data analysis through additional variations with data inputs: (i) rain gages in three counties, (ii) regionally located, but different topographically as well as different (iii) $P_{min}$ values ($P_{min}$ >0.01”, $P_{min}$ >0.05”, $P_{min}$ >0.10”, $P_{min}$ >=0.01”, $P_{min}$ >=0.05”, $P_{min}$ >=0.10”) and (iv) Ts values (greater than, or greater than or equal to, 6-, 8-, 12-, 24-, 40-hours).

Step 6) The collective of model outputs represent the total number of storm events and sensitivity of the results as a function of one-hour rain gages, minimum storm depth for precipitation events, and duration between storm events. Additional statistical analysis and binning of results provide better insight into the storm events seasonally.

Step 7) Based on one-hour precipitation depths across three counties along the Front Range of Colorado, the WQCV precipitation depth of 0.60 inches was validated with the same analysis. However, statistical results can vary based on additional factors such as period of record, location, or other data reporting elements. The data reporting elements highlight hourly precipitation analysis limitations to using hourly data for defining storm events. Variability in results trace to conditional data elements such report depths (significant digits) and reporting intervals of the rainfall data since these are not sufficient to investigate the precipitation intensity. For example, a precipitation value reports a total of 0.8 inches at a given hour marker; however, it is not possible to determine the distribution of how that 0.8 inches fell in the previous hour (all 0.8 inches could have fallen within the first 10 minutes), which results in a different intensity outside of what is used to define the regional design storm. The hourly precipitation depths, reported at an hourly interval can be limited when analyzing sub-hourly precipitation characteristics. Therefore, one-hour precipitation data were also limited to develop a water quality intensity or corresponding peak flow as additional data should be used to evaluate and assess corresponding water quality events.

Step 8) After determining the one-hour precipitation datasets were limited to select conditions to investigate precipitation intensity, additional literature and data collection was completed to obtain other regional data reporting at sub-hourly intervals (and depths).

Step 9) The first data obtained as a result of phase two in the data collection effort was not a dataset, but rather a data analysis, UDFCDPeakRainIDF_1985-2019.xlsx, created by MHFD and WET. The data analysis workbook presents all precipitation events measured at ALERT rain gages for a range of durations that are analyzed and categorized using NOAA Atlas 14 intensity duration data. A total of 242 rain gages are used in the analysis. Durations and return periods range from 5-minute to 24-hour and from the 2-year to 1000-year, respectively. Data and data analyses were analyzed to investigate minor storm events as a function of the individual rain gage data analysis, return period and duration. These included only focusing on all of the storm durations less than or equal to an hour and return periods more frequent than the 10-year. Conversely, these data were also investigated from the total storm depth by focusing on the more probable storms with less than one inch of rainfall. Statistics on the different return period were used to develop different intensity duration values for the corresponding regional rain gages.

Step 10) The second dataset obtained for phase two included precipitation data collected via NOVASTAR5 by location, time period, and reporting time interval. A total of eight additional stations were selected and data were collected at a 5-minute reporting time interval. A period of 1/2017 through 5/2021 was used to represent and compare results against independent datasets. An important note is that these datasets, which are all reported at a 5-minute interval, had different minimum reporting depths (0.01" vs. 0.04") and included seasonally variable operations (some are not operated in the winter months).

Step 11) The algorithm developed in step 3 was then applied to the 5-minute precipitation depths collected from the seven stations. Model inputs function in the similar manner as the one-hour precipitation depths; however, now the reporting intervals can be further investigated using sub-hourly and corresponding intensity duration can be more accurately quantified for those events with short storm durations. In addition to the statistical analysis, a storm event database was developed for these stations and corresponding years. Since the precipitation data is collected at a 5-minute interval, a storm event database provided a means to normalize the storm depth versus the storm duration that could then be used to illustrate real storm events distributions against current design storm distributions. The different storm distributions for the region were used to develop a baseline and resulting storms were then plotted against the regional distribution.

**In most cases, the current design storm distribution reasonably captures the majority of storm events as most of the storm events are bounded by the two distribution curves.**

Step 12) Unlike previous steps, the last part of the WQE analysis considered a collection of paired rainfall and runoff data from active MHFD stormwater monitoring sites to better validate the results from previous methods and other accepted hydrologic routing methods. Hydrologic routing methods such as rational method, TR-55, CUHP, CUHP-SWMM, or SWMM among others, provide limited insight to flow regime and often produce widely varying results depending on various inputs and factors of design (scale, imperviousness, time of concentration, etc.).

To validate the rainfall intensity values are appropriate, independent data were collected from two stormwater monitoring sites with active telemetry setup. The sites provided maximum rainfall intensities based on a similar algorithm developed and internally coded into the monitoring software to report intensities after storm events. Telemetry sites connect and report data collected via an ISCO tipping bucket rain gage as well as flow monitoring equipment such as weirs with pressure transducers. Real storm events and real runoff events measured via a flow were compared for the different intensities to those from design methods.

Step 13) Summarized the results.

## Precipitation Dataset #1: NCEI CDO Stations with Hourly Precipitation Records

| COUNTY | STATION ID (COOP: #) | STATION NAME | # OF RECORDS | PERIOD OF RECORD |
|---|---|---|---|---|
| Denver County | 052225 | DENVER WEATHER SERVICE OFFICE CITY CO US | 8838 | 8/1/1948 - 2/1/1975 |
| | 052211 | DENVER INTERNATIONAL AIRPORT CO US | 6347 | 3/1/1995 - 12/29/2013 |
| | 052220 | DENVER STAPLETON CO US | 20209 | 8/1/1948 - 12/14/2013 |
| Jefferson County | 051826 | CONIFER 3 NE CO US | 3602 | 2/1/1965 - 4/20/1981 |
| | 052633 | ELK CREEK CO US | 6351 | 8/1/1948 - 3/1/1965 |
| | 052790 | EVERGREEN CO US | 6703 | 1/1/1968 - 1/1/2014 |
| | 052795 | EVERGREEN 2 SW CO US | 3375 | 8/1/1948 - 2/1/1968 |
| | 053386 | GOLDEN 3 S CO US | 4574 | 5/1/1976 - 1/1/2014 |
| | 054293 | INTER CANYON CO US | 5105 | 5/1/1981 - 1/1/2014 |
| | 055765 | MORRISON 1 SW CO US | 11575 | 1/1/1958 - 1/1/2014 |
| | 055777 | GOLDEN 3 S CO US | 105 | 2/1/1975 - 6/1/1976 |
| | 055805 | MORRISON 1 SW CO US | 3020 | 8/1/1948 - 12/7/1957 |
| | 058994 | WHEAT RIDGE CO US | 1221 | 4/1/1975 - 7/1/1983 |
| Boulder County | 057648 | SILVER LAKE CO US | 19455 | 8/1/1948 - 1/1/2014 |
| | 055881 | NEDERLAND 5 NNW CO US | 14476 | 8/1/1948 - 12/31/2013 |
| | 050183 | ALLENSPARK 2 SE CO US | 10776 | 8/1/1948 - 12/4/2013 |
| | 055121 | LONGMONT 6 NW CO US | 3821 | 10/1/1996 - 12/30/2013 |
| | 050843 | BOULDER 2 CO US | 2964 | 1/1/1977 - 9/1/1996 |

## Precipitation Dataset #2: ALERT Rain Gage Network Stations

| Station ID | Station Name | Station Elevation | Reporting Precip. Type | Reporting Notes |
|---|---|---|---|---|
| 10020 | OneRain Weather Station | ~4947 | 0.01" | Located in Longmont (OneRain station) |
| 1480 | Third Creek at DIA | 5179 | 0.04" | March to November only |
| 920 | Aurora Town Hail | 5464 | 0.04" | Reporting interval 0.04"; time interval < 5 minutes |
| 4360 | Boulder Justice Center | 5382 | 0.04" | All rainfall from water years (2017-2021); missing data for 1/2020 to 6/2020 |
| 2370 | Red Rocks Park | 6104 | 0.04" | |
| 1350 | Chatfield Dam | 5545 | 0.04" | |
| 2750 | Castle Rock | 6488 | 0.04" | |
| *NOTE: Not all stations report at the same precipitation depth intervals. Not all stations report throughout the entire season (i.e. no winter data collection; varies by year). | | | | |

## Precipitation Dataset #3: Active Stormwater Monitoring Sites

| Station ID | Station Name | Elevation (ft) | Reporting Notes | Reporting Precip. Type |
|---|---|---|---|---|
| 20030 | River Run Park | ~5282 | BMP monitoring site rain gage; 5-minute; 0.01-in | 0.01" |
| 20010 | MHFD-Industry-NE-WQ-Sand | ~5280 | BMP monitoring site rain gage; 5-minute; 0.01-in | 0.01" |
| 20020 | MHFD-Industry-SW-WQ | ~5280 | BMP monitoring site rain gage; 5-minute; 0.01-in | 0.01” |

## Precipitation Analysis #1: Storm Event with Statistics and Binning Methods

- Method uses hourly precipitation data and sub-hourly precipitation datasets.
- Storm event statistics determined based on a set of paired conditions to define a precipitation intensity with respect to storm event depths. A binning approach is applied for values less than 0.50 inches.
- When data interval reporting is hourly, sub-hourly precipitation analysis may be undetermined due to the nature of the reporting to statistical analysis (i.e. can only look at time interval at 60 minutes – storm characteristics during the one hour block are not uniform).

Bin Name: Stapleton Airport

| Intensity Range | | Number of Events | Avg. Intensity [in/hr] | Average Rainfall Depth [in] | Events/Year |
|---|---|---|---|---|---|
| Low [in/hr] | High [in/hr] | | | | |
| 0.01 | 0.5 | 1841 | 0.16 | 0.50 | 40.91 |
| 0.51 | 1 | 141 | 0.70 | 1.48 | 3.13 |
| 1.01 | 1.5 | 39 | 1.23 | 2.10 | 0.87 |
| 1.51 | 2 | 18 | 1.74 | 2.55 | 0.4 |
| 2.01 | 2.5 | 5 | 2.32 | 3.79 | 0.11 |
| 2.51 | 3 | 3 | 2.75 | 3.11 | 0.07 |
| 3.01 | 3.5 | 3 | 3.17 | 3.63 | 0.07 |

Bin Name: Stapleton Airport; only events less than 0.50”

| Intensity Range | | Number of Events | Avg. Intensity [in/hr] | Average Rainfall Depth [in] | Events/Year |
|---|---|---|---|---|---|
| Low [in/hr] | High [in/hr] | | | | |
| 0.01 | 0.1 | 726 | 0.07 | 0.22 | 16.13 |
| 0.11 | 0.2 | 617 | 0.15 | 0.49 | 13.71 |
| 0.21 | 0.3 | 284 | 0.25 | 0.75 | 6.31 |
| 0.31 | 0.4 | 146 | 0.35 | 1.14 | 3.24 |
| 0.41 | 0.5 | 68 | 0.46 | 1.26 | 1.51 |

## Precipitation Analysis #2: Storm Event Statistics with Regional Analysis Summary Workbook

- Method uses previously completed annual summaries (ALERT Gage Analysis).
- The source of the information is the UDFCD Peak Intensity 1985-2020 Data (available as Excel document).
- Analysis represents Microsoft Excel's descriptive statistics of 2-year peak intensities at 252 sites.

Descriptive Statistics for 2-YR Peak Intensities

| ***Statistic*** | ***F2yr05m*** | ***F2yr10m*** | ***F2yr15m*** | ***F2yr30m*** | ***F2yr60m*** |
|---|---|---|---|---|---|
| **Mean** | 0.265 | 0.388 | 0.473 | 0.643 | 0.795 |
| **Standard Error** | 0.0011 | 0.0016 | 0.0019 | 0.0025 | 0.0032 |
| **Median** | 0.264 | 0.387 | 0.472 | 0.644 | 0.796 |
| **Mode** | 0.252 | 0.369 | 0.450 | 0.686 | 0.846 |
| **Standard Deviation** | 0.017 | 0.025 | 0.030 | 0.040 | 0.050 |
| **Sample Variance** | 0.00028 | 0.00061 | 0.00091 | 0.00163 | 0.00252 |
| **Kurtosis** | 0.73 | 0.74 | 0.76 | -0.81 | -0.78 |
| **Skewness** | 0.52 | 0.52 | 0.52 | -0.23 | -0.18 |
| **Range** | 0.101 | 0.149 | 0.181 | 0.195 | 0.251 |
| **Minimum** | 0.228 | 0.333 | 0.407 | 0.546 | 0.669 |
| **Maximum** | 0.329 | 0.482 | 0.588 | 0.741 | 0.92 |
| **Count** | 252 | 252 | 252 | 252 | 252 |
| **Confidence Level (90.0%)** | 0.001754 | 0.002574 | 0.003134 | 0.0042035 | 0.0052166 |

## Precipitation Analysis #3: Normalized Storm Events (Reprint from Guo, 2021)

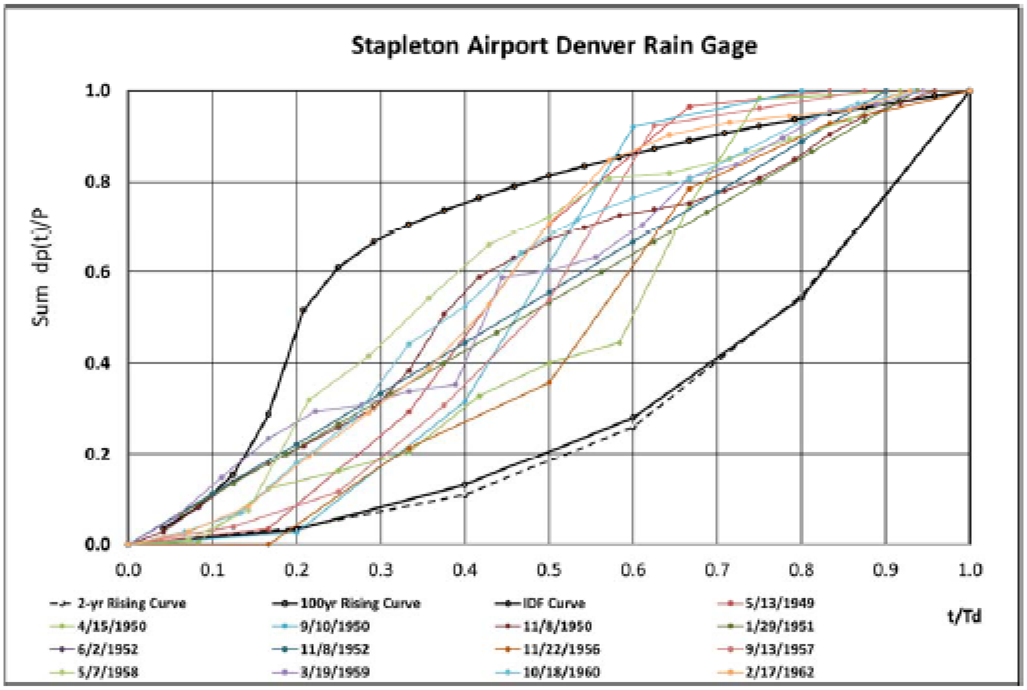


*Figure 5 Normalized Rainfall Events Recorded at Denver Stapleton Airport*

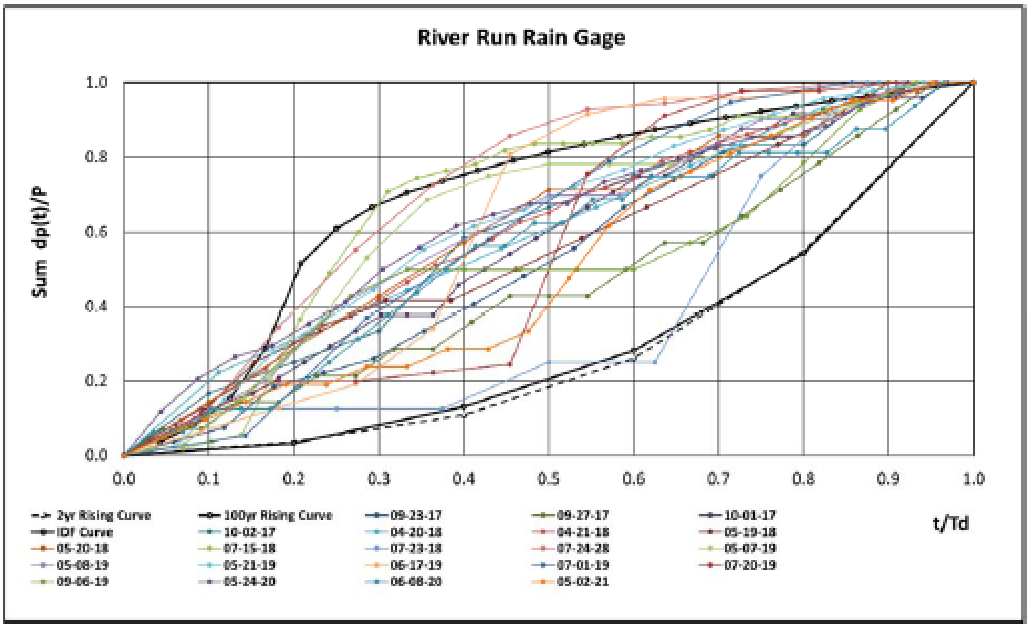


*Figure 6 Normalized Rainfall Events Recorded at Englewood River Run*

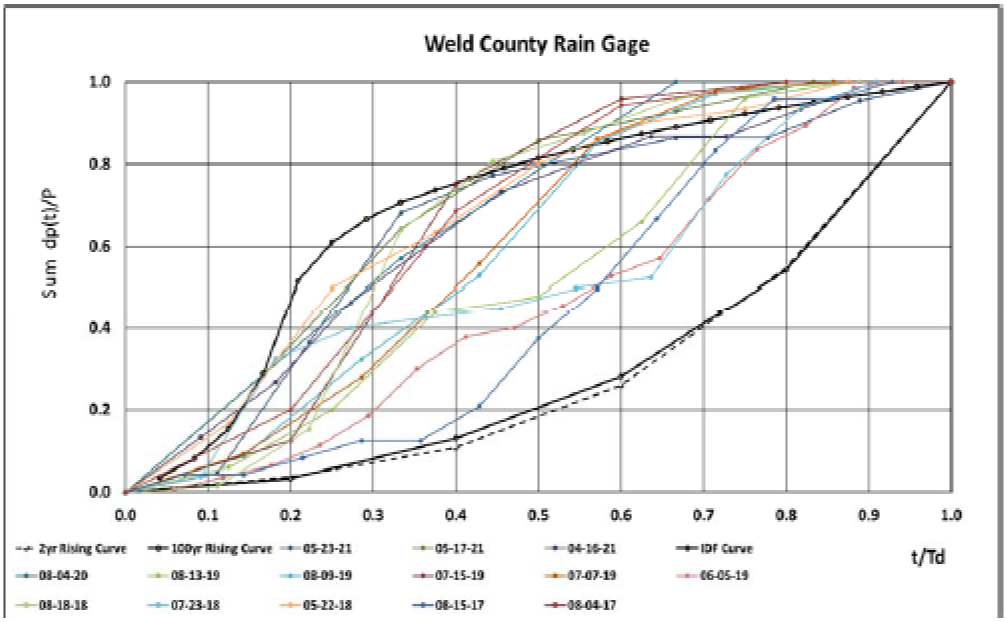


*Figure 7 Normalized Rainfall Events Recorded at Weld County*

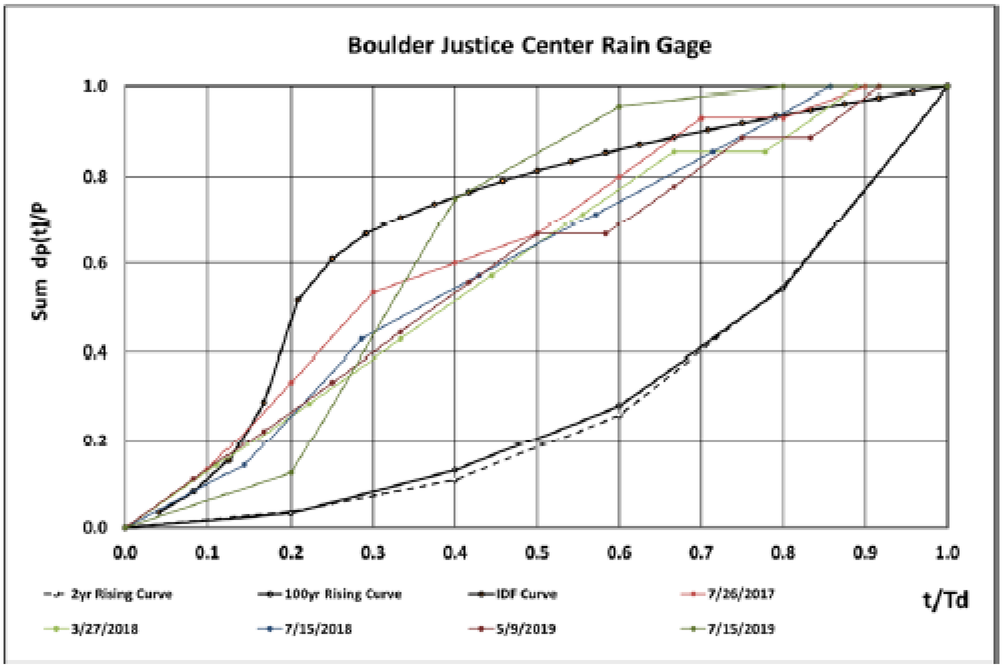


*Figure 8 Normalized Rainfall Events Recorded at Denver Stapleton Airport*

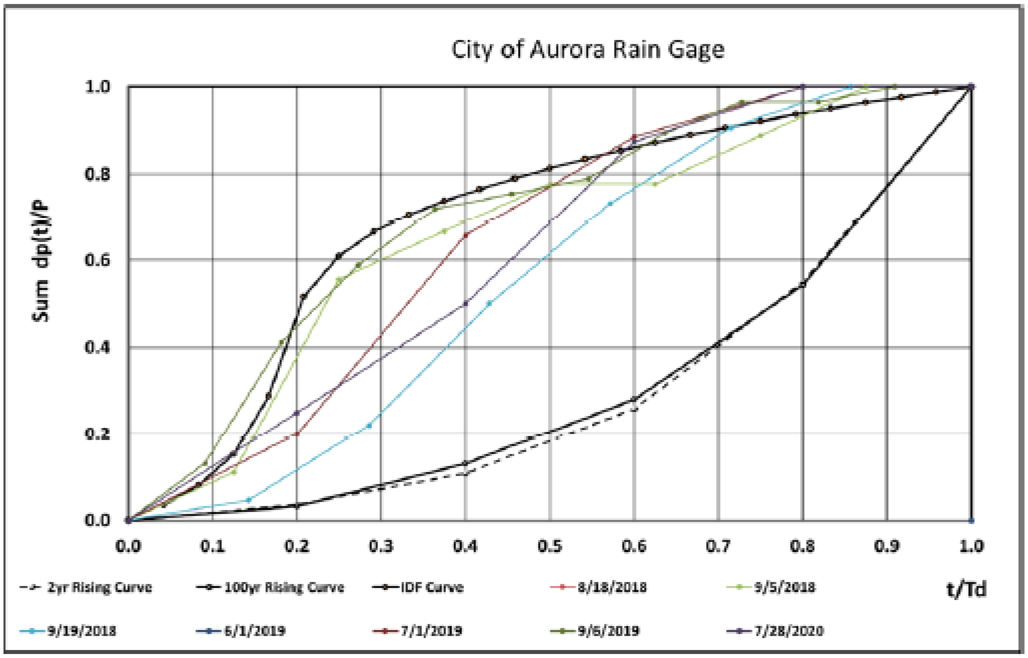
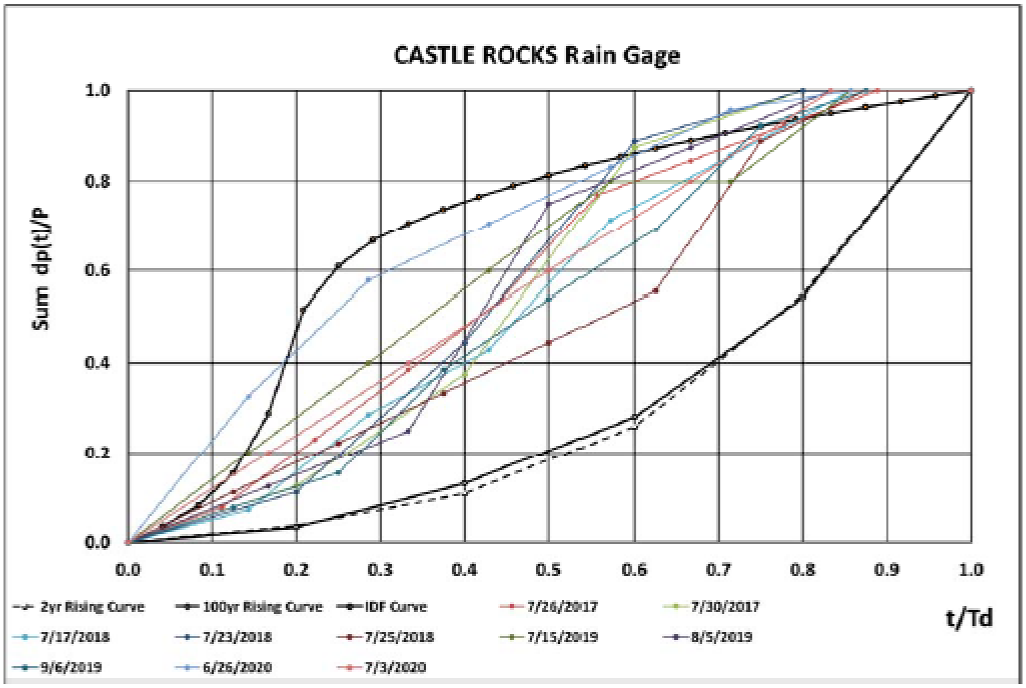


*Figure 9 Normalized Rainfall Events Recorded at City of Aurora*



## Precipitation Analysis #4: Storm Event Characteristics with Real-Time Monitoring Datasets

| Site Name | Storm Event Date(s) | Rainfall (in) | Max. 5-min Rainfall Intensity (in/hr) |
|---|---|---|---|
| RIVERRUN | 5/3/2021 | 0.95 | 0.6 |
| RIVERRUN | 5/9/2021 | 0.24 | 0.6 |
| RIVERRUN | 5/10/21 – 5/11/21 | 0.25 | 0.6 |
| RIVERRUN | 5/17/21 – 5/18/21 | 0.16 | 0.6 |
| RIVERRUN | 5/18/21 (B) | 0.10 | 0.6 |
| RIVERRUN | 5/23/2021 | 0.22 | 0.6 |
| RIVERRUN | 5/30/2021 | 0.21 | 0.6 |
| RIVERRUN | 5/31/2021 | 0.16 | 0.6 |
| RIVERRUN | 6/26/2021 | 1.29 | 54.6 (ERROR) |
| RIVERRUN | 6/29/2021 | 0.14 | 0.6 |
| RIVERRUN | 7/1/2021 | 0.39 | 1.2 |
| RIVERRUN | 7/6/21 – 7/7/21 | 0.25 | 2.4 |
| RIVERRUN | 7/14/21-7/15/21 | 0.25 | 1.8 |
| RIVERRUN | 8/14/21-8/15/21 | 0.27 | 4.2 |
| INDUSTRY-NE | 5/17/21 – 5/18/21 | 0.28 | 1.2 |
| INDUSTRY-NE | 5/18/2021 | 0.14 | 0.6 |
| INDUSTRY-NE | 5/23/2021 | 0.24 | 1.2 |
| INDUSTRY-NE | 5/25/2021 | 0.09 | 0.6 |
| INDUSTRY-NE | 5/30/2021 | 1.17 | 1.2 |
| INDUSTRY-NE | 5/31/2021 | 0.23 | 0.6 |
| INDUSTRY-NE | 6/21/2021 | 0.19 | 0.6 |
| INDUSTRY-NE | 6/25/21 – 6/26/21 | 0.36 | 0.6 |
| INDUSTRY-NE | 7/1/21 – 7/2/21 | 0.24 | 0.6 |
| INDUSTRY-NE | 7/6/21 – 7/7/21 | 0.03 | 0.6 |
| INDUSTRY-NE | 7/30/21-7/31/21 | 0.07 | 0.6 |
| INDUSTRY-NE | 7/31/21-8/1/21 | 0.01 | 0.6 |
| INDUSTRY-SW | 5/10/21 – 5/11/21 | 0.04 | 0.6 |
| INDUSTRY-SW | 5/17/21 – 5/18/21 | 0.19 | 0.6 |
| INDUSTRY-SW | 5/18/2021 | 0.09 | 0.6 |
| INDUSTRY-SW | 5/23/2021 | 0.17 | 1.2 |
| INDUSTRY-SW | 5/25/2021 | 0.07 | 0.6 |
| INDUSTRY-SW | 5/30/21 – 5/31/21 | 0.74 | 1.2 |
| INDUSTRY-SW | 5/31/2021 | 0.17 | 0.6 |
| INDUSTRY-SW | 6/21/2021 | 0.07 | 0.6 |
| INDUSTRY-SW | 6/25/21 – 6/26/21 | 0.29 | 1.2 |
| INDUSTRY-SW | 7/1/21 – 7/2/21 | 0.13 | 0.6 |
| INDUSTRY-SW | 7/6/21 – 7/7/21 | 0.31 | 1.2 |
| INDUSTRY-SW | 7/31/21 – 8/1/21 | 0.28 | 0.6 |

## Precipitation Analysis - Bibliography

Akan, A. O. (1989). Detention pond sizing for multiple return periods. *Journal of Hydraulic Engineering*, *115*(5), 650-664.

Akan, A. O. (2002). Sizing stormwater infiltration structures. *Journal of Hydraulic Engineering*, *128*(5), 534-537.

Balistrocchi, M., Grossi, G., & Bacchi, B. (2013). Deriving a practical analytical-probabilistic method to size flood routing reservoirs. *Advances in water resources*, *62*, 37-46.

Beck, M. B. (1987). Water quality modeling: a review of the analysis of uncertainty. *Water Resources Research*, *23*(8), 1393-1442.

Ben Urbonas, P. L., James, C. Y., Tucker, L. S., & PG, E. D. (1989). SIZING A CAPTURE VOLUME FOR STORMWATER QUALITY ENHANCEMENT.

Chahar, B. R., Graillot, D., & Gaur, S. (2012). Storm-water management through infiltration trenches. *Journal of Irrigation and Drainage Engineering*, *138*(3), 274-281.

Choe, B., Jo, D., Lee, J., & Kim, S. (2015). Design of water quality capture volume for infiltration trench. *Journal of the Korean Society of Hazard Mitigation*, *15*(4), 317-323.

Cizek, A. R., & Hunt, W. F. (2013). Defining predevelopment hydrology to mimic predevelopment water quality in stormwater control measures (SCMs). *Ecological engineering*, *57*, 40-45.

Fahui, N., Zhanmeng, L., & Haifeng, Y. (2010). Evaluating runoff capture volume of detention tank with rainfall probabilistic analysis methodology. *Water & Wastewater Engineering*, (6), 11.

Froehlich, D. C. (2009). Graphical calculation of first-flush flow rates for storm-water quality control. *Journal of irrigation and drainage engineering*, *135*(1), 68-75.

Guo, J. C. (1999). Detention basin sizing for small urban catchments. *Journal of Water Resources Planning and Management*, *125*(6), 380-382.

Guo, J. C. (2008). Volume-based imperviousness for storm water designs. *Journal of irrigation and drainage engineering*, *134*(2), 193-196.

Guo, J. C. (2008). Volume-based imperviousness for storm water designs. *Journal of irrigation and drainage engineering*, *134*(2), 193-196.

Guo, J. C., & Cheng, J. Y. (2008). Retrofit storm water retention volume for low impact development. *Journal of irrigation and drainage engineering*, *134*(6), 872-876.

Guo, J. C., & Luu, T. M. (2015). Hydrologic model developed for stormwater infiltration practices. *Journal of Hydrologic Engineering*, *20*(9), 06015001.

Guo, J. C., & Urbonas, B. (1996). Maximized detention volume determined by runoff capture ratio. *Journal of Water Resources Planning and Management*, *122*(1), 33-39.

Guo, J. C., & Urbonas, B. (2002). Runoff capture and delivery curves for storm-water quality control designs. *Journal of Water Resources Planning and Management*, *128*(3), 208-215.

Guo, J. C., Blackler, G. E., Earles, T. A., & MacKenzie, K. (2010). Incentive index developed to evaluate storm-water low-impact designs. *Journal of Environmental Engineering*, *136*(12), 1341-1346.

Guo, J. C., Li, J. Q., Urbonas, B., & Wang, W. L. (2019). Runoff Capture Methods Developed for Stormwater Low-Impact Development Designs. *Journal of Hydrologic Engineering*, *24*(4), 04019005.

Guo, J. C., Urbonas, B., & MacKenzie, K. (2014). Water quality capture volume for storm water BMP and LID designs. Journal of Hydrologic engineering, 19(4), 682-686.

Guo, J., & Hsu, S. C. E. (2010). Normalized Runoff Capture Volumes for Low Impact Designs. In *Low Impact Development 2010: Redefining Water in the City* (pp. 475-483).

Harmel, R. D., Cooper, R. J., Slade, R. M., Haney, R. L., & Arnold, J. G. (2006). Cumulative uncertainty in measured streamflow and water quality data for small watersheds. *Transactions of the ASABE*, *49*(3), 689-701.

Heaney, J. P. (1986). Research needs in urban storm-water pollution. *Journal of Water Resources Planning and Management*, *112*(1), 36-47.

Heitz, L. F., Khosrowpanah, S., & Nelson, J. (2000). SIZING OF SURFACE WATER RUNOFF DETENTION PONDS FOR WATER QUALITY IMPROVEMENT 1. *JAWRA Journal of the American Water Resources Association*, *36*(3), 541-548.

Heitz, L. F., Khosrowpanah, S., & Nelson, R. J. (1996). *Evaluation of Storm Water Capture Volume for Water Quality Management* (No. 80). Technical Report.

Hsu, S. C. E., Guo, J., & Fultz, R. (2010). Storm water quality control volume for Southwest Region of USA. In *Low Impact Development 2010: Redefining Water in the City* (pp. 1397-1405).

Jia, Z., Tang, S., Luo, W., Li, S., & Zhou, M. (2016). Small scale green infrastructure design to meet different urban hydrological criteria. *Journal of Environmental Management*, *171*, 92-100.

Jones, J. E., Earles, T. A., Fassman, E. A., Herricks, E. E., Urbonas, B., & Clary, J. K. (2005). Urban storm-water regulations—are impervious area limits a good idea?.

Kerian, K. Effect of Minimum Inter Event Time on Water Quality Capture Volume. *JPS*, *5522047*, 5331048.

Kim, S., & Jo, D. J. (2007). Runoff capture curve for non-point source management. *Journal of Korean Society on Water Environment*, *23*(6), 829-836.

Lee, D. H., Min, K. S., & Kang, J. H. (2014). Performance evaluation and a sizing method for hydrodynamic separators treating urban stormwater runoff. *Water science and technology*, *69*(10), 2122-2131.

Lee, J. G., & Heaney, J. P. (2003). Estimation of urban imperviousness and its impacts on storm water systems. *Journal of Water Resources Planning and Management*, *129*(5), 419-426.

Lenhart, J. H. (2004). Methods of sizing water quality facilities. *Stormwater*, *5*(4), 4-7.

Loáiciga, H. A., Sadeghi, K. M., Shivers, S., & Kharaghani, S. (2015). Stormwater control measures: optimization methods for sizing and selection. *Journal of Water Resources Planning and Management*, *141*(9), 04015006.

Loganathan, G. V., Watkins, E. W., & Kibler, D. F. (1994). Sizing storm-water detention basins for pollutant removal. *Journal of Environmental Engineering*, *120*(6), 1380-1399.

McCuen, R. H., & Moglen, G. E. (1988). Multicriterion stormwater management methods. *Journal of Water Resources Planning and Management*, *114*(4), 414-431.

Muthanna, T. M., Viklander, M., & Thorolfsson, S. T. (2007). An evaluation of applying existing bioretention sizing methods to cold climates with snow storage conditions. *Water science and technology*, *56*(10), 73-81.

Nehrke, S. M., & Roesner, L. A. (2004). Effects of design practice for flood control and best management practices on the flow-frequency curve. *Journal of Water Resources Planning and Management*, *130*(2), 131-139.

Palhegyi, G. E. (2010). Modeling and sizing bioretention using flow duration control. *Journal of Hydrologic Engineering*, *15*(6), 417-425.

Park, D., Song, Y. I., & Roesner, L. A. (2013). Effect of the seasonal rainfall distribution on storm-water quality capture volume estimation. *Journal of Water Resources Planning and Management*, *139*(1), 45-52.

Phelps, S. C. (2005). Maximum extent practical stormwater treatment utilizing ASCE water quality capture volume methodology. In *Managing Watersheds for Human and Natural Impacts: Engineering, Ecological, and Economic Challenges* (pp. 1-10).

Sadeghi, K. M., Loáiciga, H. A., & Kharaghani, S. (2018). Stormwater control measures for runoff and water quality management in urban landscapes. *JAWRA Journal of the American Water Resources Association*, *54*(1), 124-133.

Sansalone, J. J., & Teng, Z. (2005). Transient rainfall-runoff loadings to a partial exfiltration system: Implications for urban water quantity and quality. *Journal of Environmental Engineering*, *131*(8), 1155-1167.

Sharifi, S., Massoudieh, A., & Kayhanian, M. (2011). A Stochastic Stormwater Quality Volume-Sizing Method with First Flush Emphasis. *Water environment research*, *83*(11), 2025-2035.

Sullivan, D., & Borst, M. (2004). Research in Urban Stormwater BMPs. In *Watershed Management* (pp. 1-10).

Tang, S., Xu, Q., Jia, Z., Luo, W., & Shao, Z. (2019). Estimating Errors in Sizing LID Device and Overflow Prediction Using the Intensity-Duration-Frequency Method. *Water*, *11*(9), 1853.

Urbonas, B. (1994). Assessment of stormwater BMPs and their technology. *Water Science and Technology*, *29*(1-2), 347-353.

Urbonas, B., & DENVER, C. (1979, July). Reliability of design storms in modeling. In *Proceedings of the International Symposium on Urban Storm Runoff, University of Kentucky, Lexington, Ky* (pp. 23-26).

Urbonas, B., & Wulliman, J. (2007). Stormwater runoff control using full spectrum detention. In *World Environmental and Water Resources Congress 2007: Restoring Our Natural Habitat* (pp. 1-8).

Urbonas, B., Guo, J. C., & Tucker, L. S. (1990). Optimization of stormwater quality capture volume. In *Urban Stormwater Quality Enhancement: Source Control, Retrofitting, and Combined Sewer Technology* (pp. 94-110). ASCE.

Wulliman, J., & Urbonas, B. (2007). Peak flow control for a full spectrum of design storms. *NOVATECH 2007*.

Zhang, K., Che, W., Zhang, W., & Zhao, Y. (2016). Discussion about initial runoff and volume capture ratio of annual rainfall. *Water Science and Technology*, *74*(8), 1764-1772.