# DEVELOPING PROTOCOLS FOR CONSISTENCY IN CUHP/SWMM HYDROLOGY FOR LARGE DISCRITIZED CATCHMENTS

Derek N. Rapp, P.E., CFM and Ben R Urbonas, P.E., D.WRE

For Ken A MacKenzie, P.E.

Manager, Master Planning Program

## Urban Drainage and Flood Control District

# DEVELOPING PROTOCOLS FOR CONSISTENCY IN CUHP/SWMM HYDROLOGY FOR LARGE DISCRITIZED CATCHMENTS

Derek N. Rapp, P.E., CFM[1] and Ben R Urbonas, P.E., D.WRE[2]

## INTRODUCTION

Early master planning (MP) and flood hazard area delineation (FHAD) projects (i.e., prior to 1978) were performed by the Urban Drainage and Flood Control District (UDFCD) using the Colorado Urban Hydrograph Procedure (CUHP) with limited, or no routing of resulting storm hydrographs through a network of conveyances. Also, the CUHP model used then differed substantially from the one used today. In 1979 a new version of CUHP was developed that was calibrated to the rainfall-runoff data collected within UDFCD boundaries and nearby areas. As technology progressed, hydraulic routing through conveyance elements was used more and more. Eventually UDFCD adopted the U.S. Corps of Engineers Missouri Region's version of the U.S. EPA Runoff Block of the Storm Water Management Model (SWMM). This became the standard for hydraulic routing through a system of conveyance elements whenever hydrology studies for MP and FHAD projects were performed. As technology progressed further, this routing model was replaced by the currently used EPA SWMM 5 model.

Over a number of past years an issue emerged whenever hydrology was being developed for projects within the UDFCD. It was often observed that the peak discharges for various design storms became higher in downstream reaches of the catchments as the density of catchment discretization increased. This appeared to be the result of using many small sub-catchments for CUHP runs instead of only using a few large sub-catchments. The trend to use more intense discretization continued and some of these smaller sub-catchments being used are less than 10-acres in size, whereas in the past they may have been two-hundred acres, four-hundred acres or greater than one square mile in size.

The current CUHP model was calibrated using data from catchments ranging from a little over 90-acres to 3.1-square miles in size. CUHP quantifies the rate of runoff response on the basis of the catchment's imperviousness (Ia), slope (So) and shape, namely length (L) and length to centroid (Lca). As a result, it accounts for the runoff routing and travel time through the catchment. As the catchment gets larger, the time to peak (Tp) increases and its unit discharge decreases. The current CUHP model, by its parametric nature, reflects the timing associated with the surface and conveyance routing through the catchments, consistent with when it was originally calibrated.

---

[1] Peak Stormwater Engineering, LLC, Louisville, Colorado
[2] Urban Watersheds, LLC, Denver, Colorado

What has been observed is that whenever denser discretization was used (i.e., many small sub-catchments) and runoff was routed through a system of conveyance elements, the unit discharges often far exceeded the rates that would have resulted from the use of only a few large catchments. This led to inconsistencies in results for defining floodplains and in the sizing of required storm sewers, channels, culverts, bridges, etc. These inconsistencies were thought to be caused either by overestimation of runoff for small sub-catchments in CUHP or because of the differences in runoff routing between those built into CUHP and those employed by users of SWMM. In response to this, the UDFCD initiated this project to develop protocols that would reconcile the routed results for many discretized sub-catchments with the regionally-calibrated CUHP results for larger catchments.

## TESTING SENSITIVITY OF CUHP AND SWMM ROUTING

### CUHP Sensitivity to Catchment Size

UDFCD provided information on several recently master planned catchments to help develop these protocols. However, as a first step, the sensitivity of CUHP to discretization density was tested using idealized larger catchments. Since the current CUHP user guidance recommend that this method be used for catchments up to 5.0 square miles (3,200 acres) in area, a single catchment of this size was developed. This single catchment was then subdivided into a series of eight progressively smaller average sub-catchment sizes, ranging from 2.5- down to 0.025-square miles (1,600- down to 16-acres). The following input parameters were kept the same for all sub-catchments; slope of 0.03 ft/ft, 40% imperviousness, depression storage of 0.3-in for pervious areas and 0.1-in for impervious areas, initial infiltration rate of 3.0 in/hr, decay rate of 0.0018 1/sec, and final infiltration rate of 0.5 in/hr. The two length parameters were proportioned by area as follows:

$$Length: L = 2 * \sqrt{Area/2} \qquad\qquad Length\ to\ centroid: L_c = 0.6 \cdot L$$

Figure 1 illustrates how the unit discharge from a catchment varies with the size of sub-catchments. There is a clear power function relationship, as expected with the standard CUHP, for areas of 0.025- to 5.0 square miles. For smaller areas, the sub-catchment response is governed by an adjustment to CUHP that was intentionally made to make results more consistent with the Rational Method.

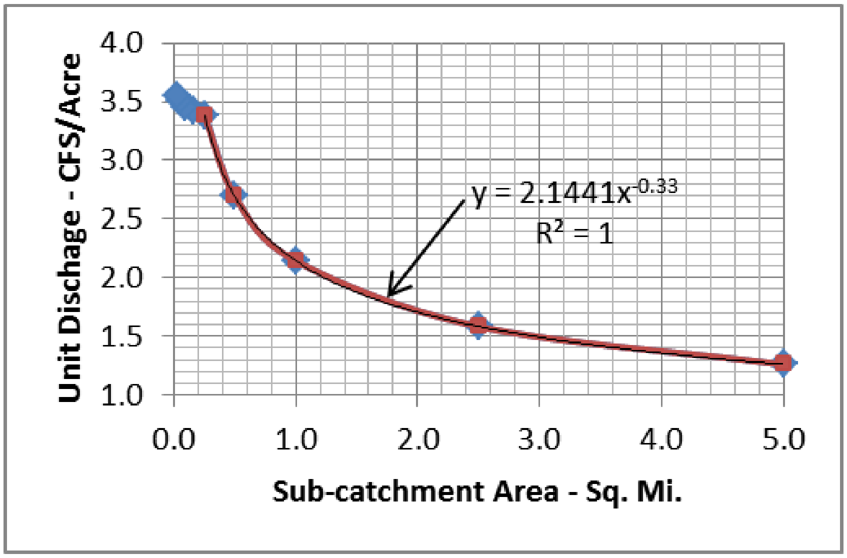


Figure 1. Unit discharge vs. Sub-catchment area for a 40% impervious idealized catchment.

**SWMM Routing Effects on CUHP Sensitivity**

All individual sub-catchment runoff hydrographs were then combined and hydraulically routed through idealized trapezoidal channels using the kinematic wave option of SWMM 5. Figure 2 illustrates the trends in peak discharge values at the outfall for various sizes of sub-catchment runoff hydrographs when routed. The channels used for routing in this example were idealized trapezoidal grass-lined channels with a 5-foot bottom width, 4:1 side slopes and a 0.01 ft/ft longitudinal slope. This was done to test sensitivity and not to represent a real world situation.

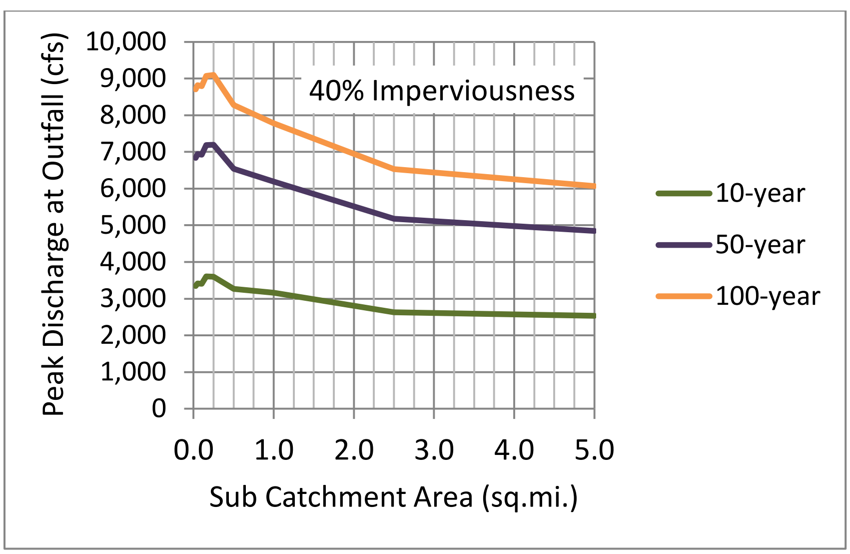


Figure 2. Hydrograph peaks at outfall for various sub-catchment sizes after routing via SWMM.

We see a progressive increase in the peak discharge at the outfall of the 5-square mile catchment as the sub-catchment density is increased, especially after sub-catchments become less than 2.5-

square miles. However, the increase in peak discharges is not as dramatic as the increase in unit discharges seen in Figure 1. This reduction in sensitivity to sub-catchment size can be attributed to the routing of the individual hydrographs through conveyance elements in SWMM and the resulting misalignment of individual sub-catchment hydrograph peaks. There appears to be a discontinuity when sub-catchment size is less than 0.25 square miles, which can be attributed to the modifications made to CUHP to make it more consistent with the Rational Method for small sub-catchments. However, further testing with other longitudinal slopes in SWMM revealed significant sensitivity to the slope parameter. This will be discussed in more detail later in this report.

### Goal of this Study

The goal of this study was to develop protocols and/or guidance on how to prevent the escalation of peak discharges that result strictly from using a denser sub-catchment discretization. As can be seen from Figures 1 and 2, the use of small, highly discretized sub-catchments result in higher discharges for large planning areas, something that is not consistent with the original finding when CUHP was calibrated against rainfall-runoff data collected in the UDFCD region. This artificial inflation of discharges results in larger flood plains and more expensive stormwater conveyance facilities.

Therefore, two approaches were looked at in parallel to limit the escalation of unit discharges for large catchments due to discretization of them into many small sub-catchments. These two approaches include:

1. Testing correction factors for adjusting the peaking coefficient Cp in CUHP
2. Testing guidance for how to set up SWMM conveyance elements that more accurately represent hydraulic routing through the catchment.

## TESTING CUHP CORRECTION FACTORS TO MINIMIZE DISCHARGE INCONSISTENCIES

In an attempt to achieve similar unit discharges for densely discretized sub-catchments to those obtained for large catchments, the peaking coefficient Cp was adjusted. The idealized catchment scenarios discussed previously were used with a 40% impervious condition as the basis for initial testing. What emerged were a set of equations that provided correction coefficients which could be applied against the Cp coefficient. Different equations were developed for planning areas of 0.5-, 1.0- 2.5- and 5.0- square miles. These same equations were then applied to catchments with 75% imperviousness which revealed a need for revised coefficients to provide unit discharges consistent with the large planning catchments. Figure 3 illustrates how the unit discharges responded to the correction coefficients for 5.0- and 1.0- square mile planning catchments with increasing density of discretization.

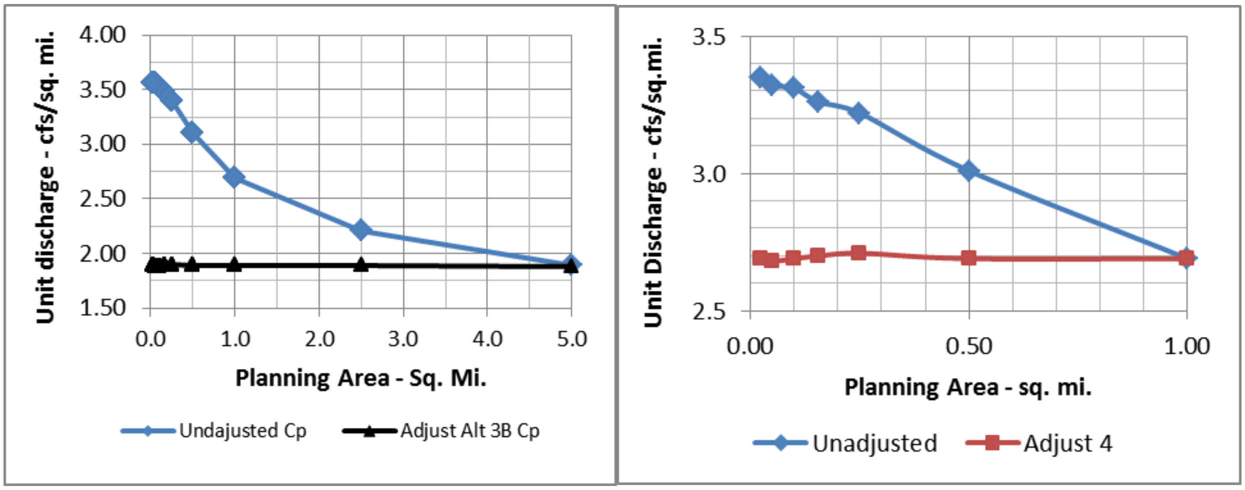


Figure 3. Unit Discharge vs. Sub-Catchment Area for 5.0-sq. mi. (left) and 1.0-sq. mi. (right) planning catchments with and without Cp adjustments.

This approach is able to produce very similar unit discharges to those obtained using the large planning area CUHP input parameters. It also produces identical runoff volumes to those produced from large planning areas. It does not, however, adjust the time to peak. The resultant runoff hydrographs were then routed using SWMM and Figure 4 illustrates the results at the outfall of the 5-square mile planning area.

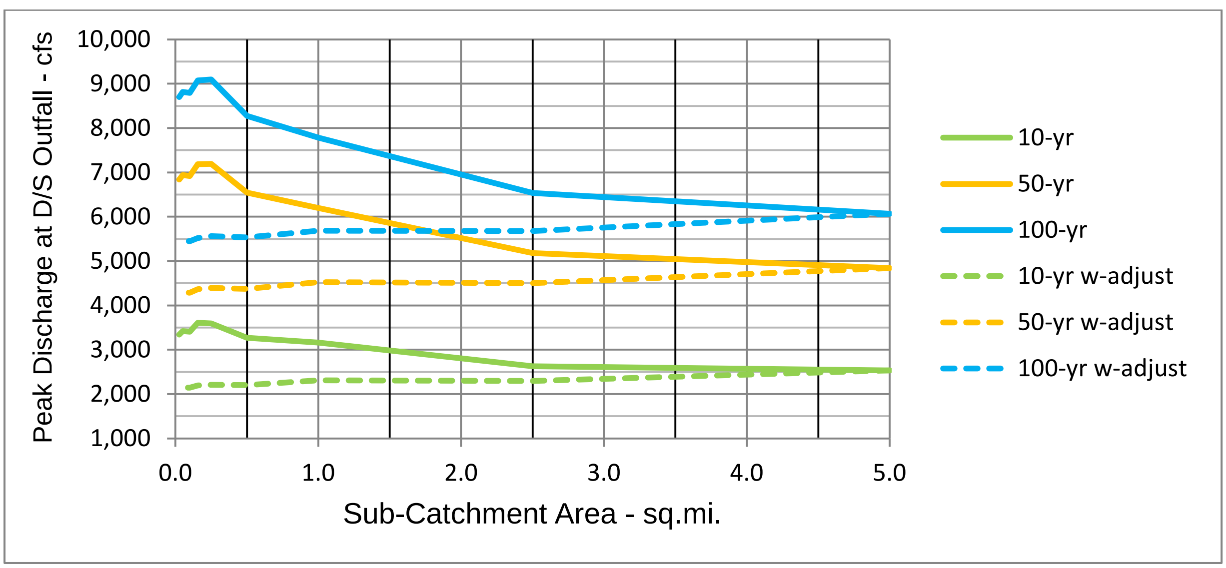


Figure 4. Cp adjusted hydrograph peak discharges at 5.0 sq. mi. catchment outfall.

As can be seen in Figure 4, the routed peak discharge results for the Cp adjusted hydrographs for various sub-catchment sizes are slightly less than the 5-square mile catchment peak discharge. Although the decrease in peak discharges are small and within confidence levels of hydrology calculations, they exist because of the minor attenuation in the SWMM routing network. If more detailed SWMM routing were used which provided greater attenuation, further adjustments would be warranted to achieve consistency.

The advantage of using this type of approach to make the unit discharges consistent for various densities of discretization is that it could be programed into the CUHP model and it would give consistent results, despite the user's capabilities.

The disadvantage of this approach includes the fact that the user may need to perform a series of adjusted Cp runs to get the appropriate peak discharges at various locations within a larger catchment. For example, in a 5-square mile catchment the user may need to make adjusted runs for 0.5- and 1.0-square mile catchments if facilities need to be sized for the upper reaches within the larger catchment. This could result in significantly more complex file handling by the planning engineer and potential for confusion by reviewers and future users.

## TESTING SWMM ROUTING PARAMETERS TO MINIMIZE DISCHARGE INCONSISTENCIES

### Using a More Realistic SWMM Conveyance System

The other attempt to achieve similar unit discharges for densely discretized sub-catchments to those obtained for large catchments was to investigate adjustments to SWMM conveyance parameters for open channels. In large catchments, open channels typically are the final routing elements for major drainageways. Modifications to SWMM input parameters such as longitudinal channel slope, channel cross-section geometry, and channel roughness were investigated to route unadjusted CUHP hydrographs obtained for various densities of catchment discretization. This was initially approached in steps to determine sensitivity as follows:

1. Developed target CUHP peak discharge hydrographs at major design points for a few idealized larger catchments ranging from 1.0- to 5.0 square miles.
2. The resultant CUHP hydrographs were used to compare results of discretization levels on downstream peaks as the individual hydrographs were routed through a simplified SWMM network of trapezoidal channels having a 10-foot bottom width and 4:1 side slopes.
3. The slope of the routing elements was changed from 0.01 ft/ft to 0.005 ft/ft to reflect potential drop structures.
4. Manning's n was increased from 0.035 to 0.050 while using the flatter 0.005 ft/ft slope.
5. A representative irregular cross-section as illustrated in Figure 5 with a longitudinal slope of 0.005 ft/ft, a channel Manning's n of 0.040 and an overbank Manning's n of 0.060 was substituted for the idealized trapezoidal channel for all the reaches.

The results of these SWMM runs used to route five 1.0-square mile and fifty 0.1-square mile CUHP storm hydrographs are illustrated in Figures 6a through 6d. They show how the outfall hydrographs shape and peak responded to variations in channel slope and Manning's n.

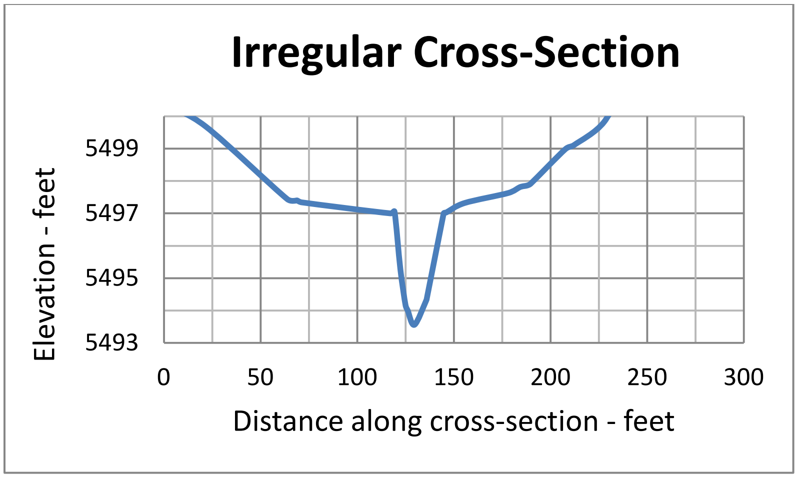


Figure 5. Representative irregular SWMM conveyance element.

The baseline condition for investigating conveyance routing was a trapezoidal channel as described earlier with a 0.01 ft/ft longitudinal slope and a Manning's n of 0.035. It is clear from analyzing Figures 6a through 6c that reducing the slope to 0.005 ft/ft and increasing Manning's n to 0.05 brought the downstream hydrograph peak for the routed densely discretized set of sub-catchments close to what a single 5-square mile catchment would produce.

Next, an irregular cross-section (see Figure 5) at a slope of 0.005 ft/ft and with a channel Manning's n of 0.040 and an overbank Manning's n of 0.060 was substituted for the trapezoidal cross-sections in this idealized 5-square mile catchment. Surprisingly, the peak discharges shown on Figure 6d were almost identical to those from a single 5-square mile catchment. It should be noted that there was a small difference in the timing of the peak, but that should not be of major concern.

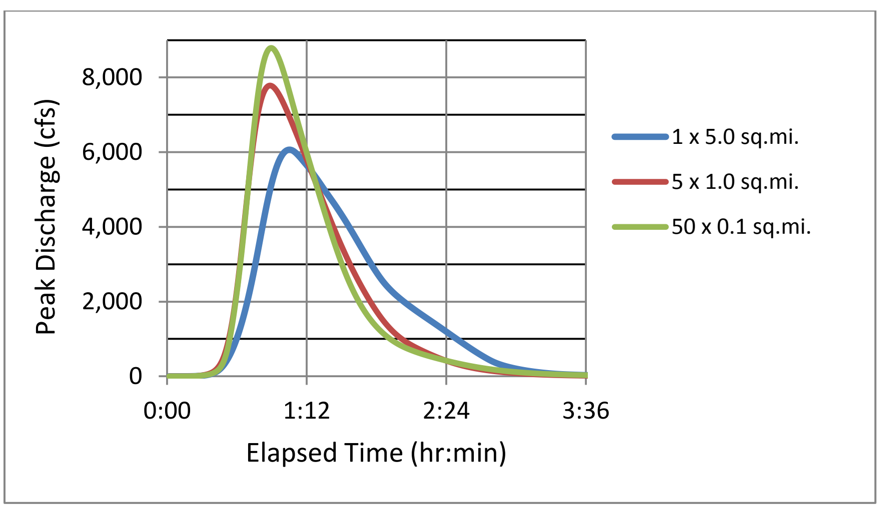


Figure 6a. Baseline trapezoidal SWMM channel with $S_o = 0.01$ & Manning's n = 0.035

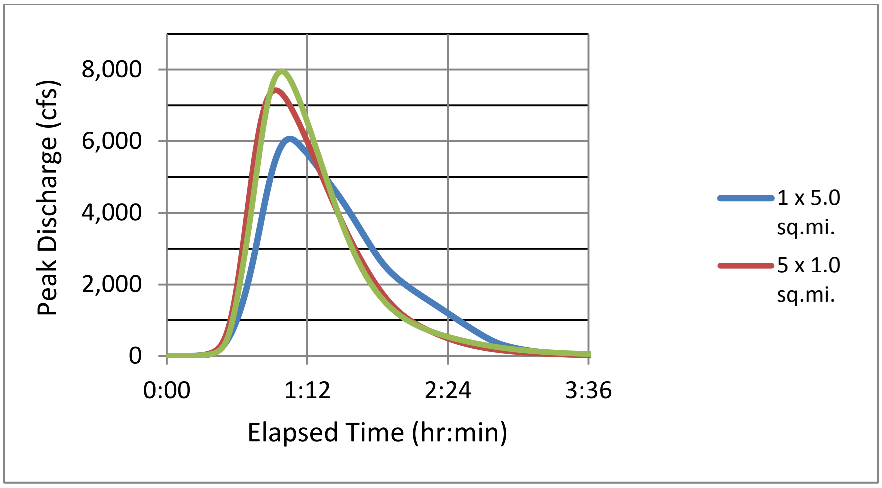


Figure 6b. Trapezoidal SWMM channel with $S_o$ = 0.005 & Manning's n = 0.035

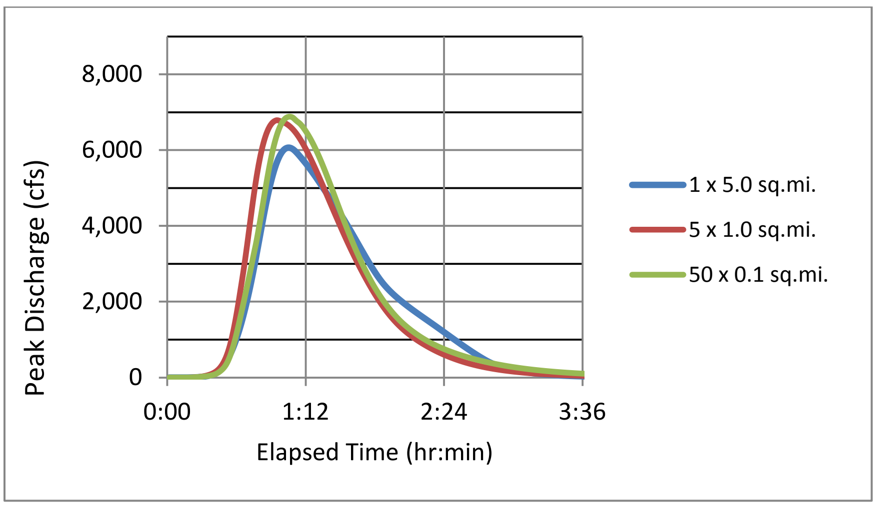


Figure 6c. Trapezoidal SWMM channel with $S_o$ = 0.005 & Manning's n = 0.050

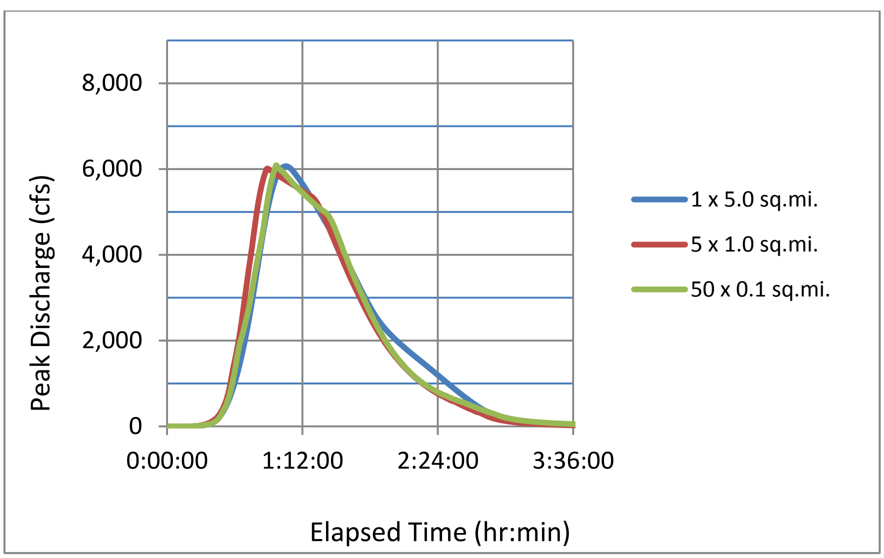


Figure 6d. Irregular channel cross-section with $S_o$ = 0.005 & Manning's n = 0.040 to 0.060

The exercise described above with an idealized catchment and experimenting with a variety of routing element parameters revealed a promising trend. What it showed was that peak discharge differences from various discretization densities can be dealt with by adopting appropriate conveyance parameters and channel geometries to better reflect the travel time through the catchment. However, adjustment of the routing element parameters and channel geometry need to remain within realistic bounds.

## TESTING SWMM ROUTING RESPONSE USING PAST MASTER PLANNING CATCHMENTS

Next the testing next progressed to the use of a few previously conducted master planning studies. The hydrologic models for the Sanderson Gulch MP, a portion of the East Toll Gate Creek MP, and a portion of the Sand Creek MP were used as the basis for testing what it takes to get more realistic peak discharges by adjusting channel routing parameters. All three had FHAD studies with HEC RAS cross-sections available which permitted the use of representative longitudinal slopes, irregular cross-sections, and roughness factors and.

### Sanderson Gulch

The Sanderson Gulch hydrology model from the master plan includes a 9.0-square mile catchment (see Figure 7) comprised of 101 sub-catchments. These 101 sub-catchments were condensed into two major sub-catchments of 4.4- and 4.6-square miles which are within the limits of applicability for CUHP. The division of these two sub-catchments is located roughly along Sheridan Blvd shown by the red line on Figure 7.

Three different CUHP-SWMM test runs were initially made. The first run was considered the baseline and used the original 101 master planning sub-catchments and original routing elements. It should be noted that all detention storage elements were removed from the original routing model so as to avoid confusion when comparing results. The second used only the two major sub-catchments along with the original routing elements and served to provide a target for peak discharges at the midpoint and at the outfall. The third used the original 101 sub-catchments with adjusted SWMM routing elements along the major drainageways of Sanderson Gulch and North Sanderson Gulch. The major conveyance element trapezoidal cross-sections were replaced by representative irregular cross-sections taken from the FHAD HEC-RAS model. The Manning’s n values from the HEC-RAS model were also used for the irregular cross-sections. Longitudinal slopes for the major conveyance elements were flattened by adding outlet offsets to reflect the effects of grade controls provided by culverts, bridges and drop structures. This differs from the original routing in that the longitudinal slopes in the original plan most often assumed a straight line between known elevations at design points and did not account for the actual channel slope between grade controls. Table 1 shows how the results for peak discharges compared.

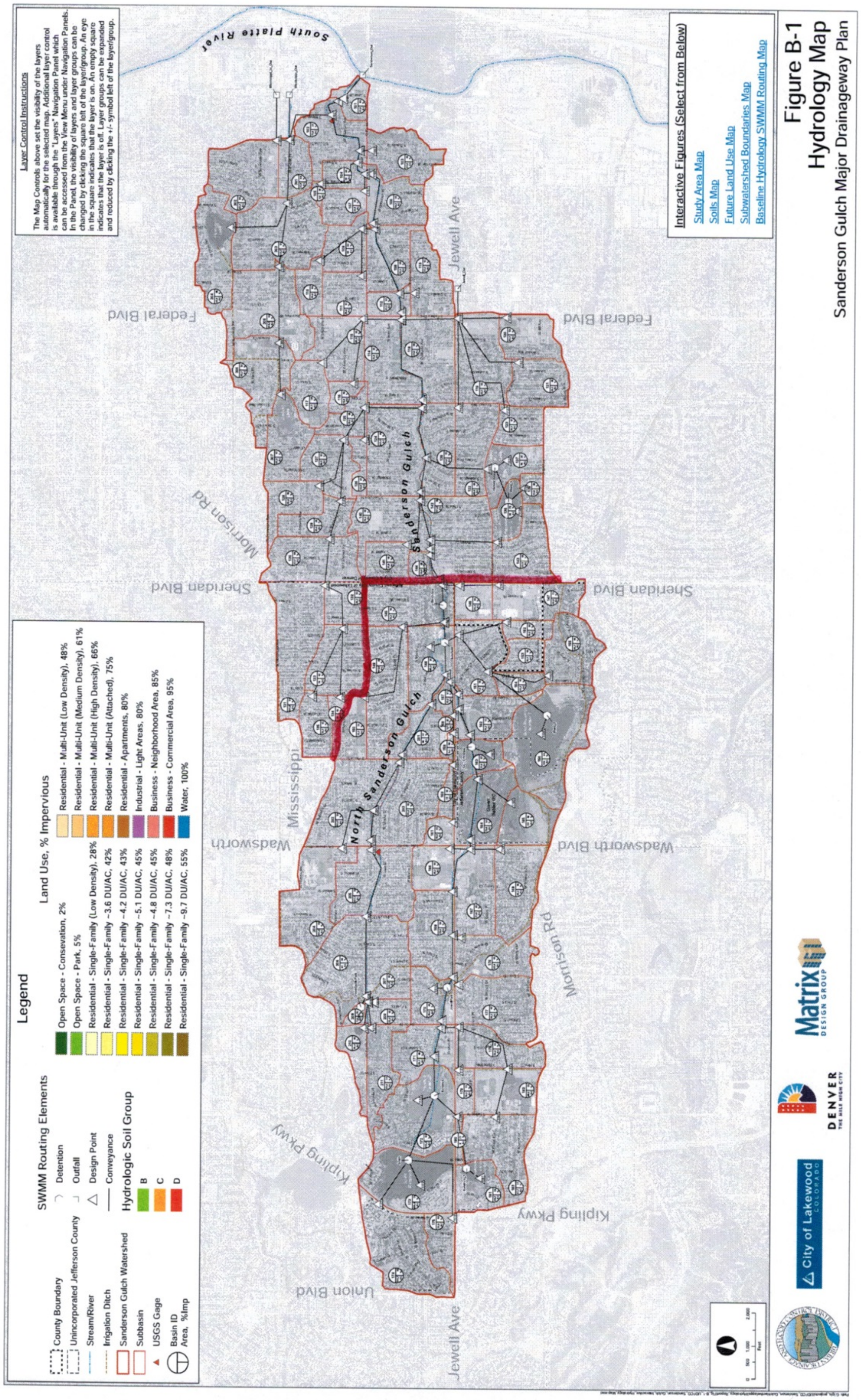


Figure 7. Catchment discretization for Sanderson Gulch hydrologic model

Table 1. Comparing MP model against 2 Sub-catchment model and Routing Adjustments MP model.

| Design Point | Original MP Model | | 2 Sub-Catchment Model | | MP Model w/ HEC-RAS Sect. & Manning's n | |
|---|---|---|---|---|---|---|
| | Peak Q cfs | Time to peak h:mm | Peak Q cfs | Time to peak h:mm | Peak Q cfs | Time to peak h:mm |
| Mid-point | **6,909** | 0:52 | **4,859** | 1:11 | **5,810** | 1:07 |
| Outfall | **11,769** | 1:09 | **8,528** | 1:31 | **8,620** | 1:28 |

The 2 Sub-Catchment model reflects what two 4.5-square mile CUHP hydrographs would produce, the outfall one resulting from combining the lower CUHP hydrograph with the mid-point hydrograph routed by the original SWMM conveyance elements. It could be considered as the target to test against how well the more densely discretized SWMM model is performing. The model with adjusted routing parameters for the major conveyance elements produced results at the outfall very close to the target model. This indicates that the increase in travel time along the major drainageways helps to spread the individual hydrographs out, thereby limiting the overlap and reducing downstream peak discharges. This effect was not as pronounced at the midpoint as expected. This is most likely because there is less major drainageway routing at this point. Also, the minor tributary routing was not adjusted as part of this test because the HEC-RAS model did not include these side tributaries. Regardless, the peak discharges for the adjusted routing model were still significantly smaller than the original MP model and were within 20% of the target peak discharges. If desired, further refinement of the adjusted routing model could be achieved by adjusting the conveyance parameters for the smaller side tributaries. Also, additional refinements may be achieved by using Manning's n values as a calibration parameter, provided that the values selected are within reasonable limits. However, the routing effects are often not that noticeable when using Manning's n as a calibration parameter. It takes major shifts in the values to produce significant changes which can lead to the problem of less defensible values.

**East Toll Gate Creek**

The Toll Gate Creek hydrology model from the master plan includes a study area of 16-square miles comprised of 112 sub-catchments (see Figure 7). The study area includes Toll Gate Creek, an inflow hydrograph representative of West Toll Gate Creek and the lower half of East Toll Gate Creek. Of this area, only the lower half of East Toll Gate Creek (8.5-square miles consisting of 62 sub-catchments) was evaluated as part of this calibration test. The inflow hydrograph for the upper half of East Toll Gate Creek and all detention storage elements were removed from the model in order to avoid confusion when comparing results. The East Toll Gate Creek (ETGC) hydrology model as shown in the upper half of Figure 8 (above the dark black line) was subdivided into 4 major sub-catchments (separated by red lines) ranging from 1.8 to 2.5 square miles.

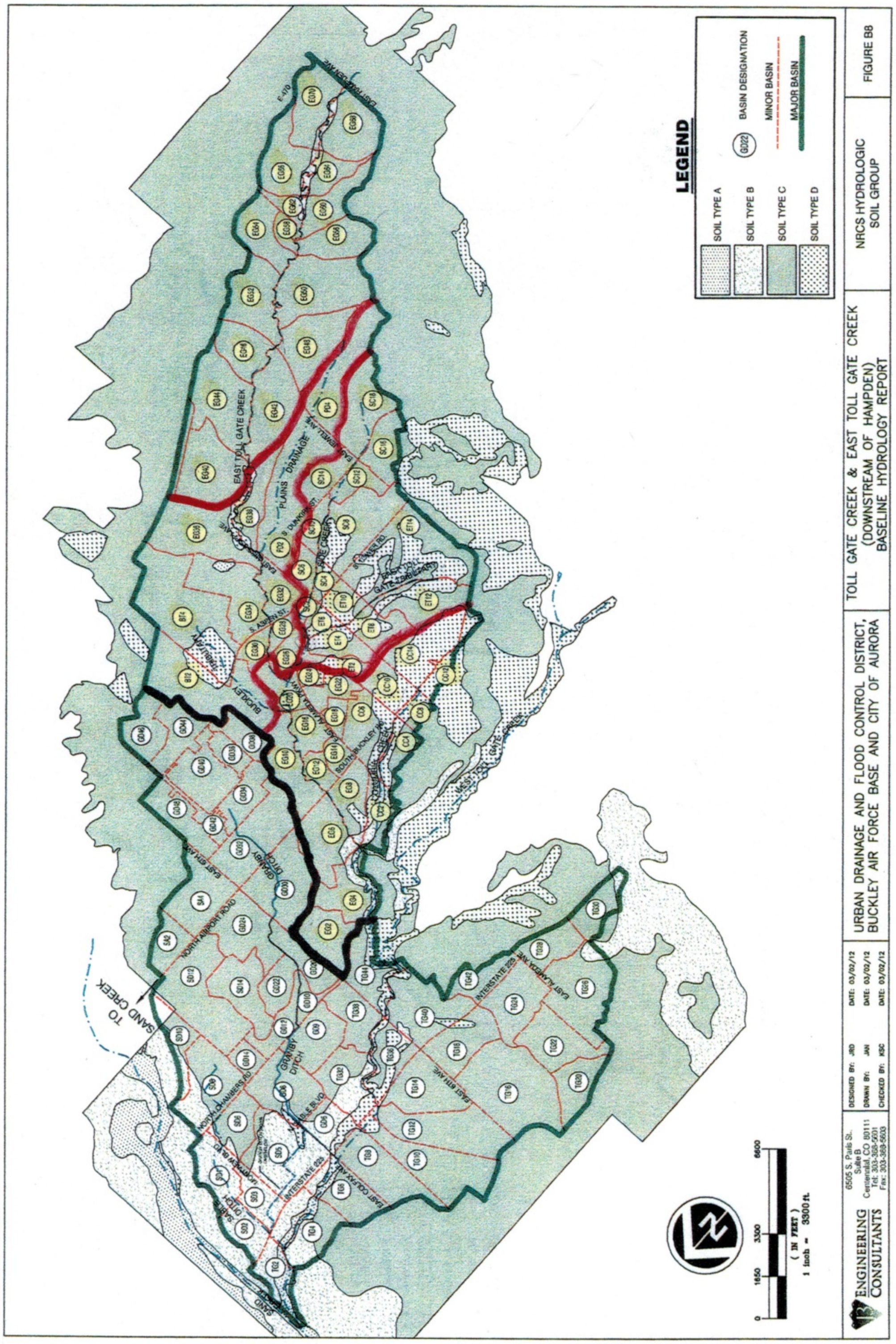


Figure 8. Catchment discretization for East Toll Gate Creek hydrologic model

Five different CUHP-SWMM models were evaluated for ETGC. The first model used the original 62 sub-catchments and routing elements from the recent master plan to represent the results typically produced in master plans. The second model used the four major sub-catchments averaging 2.1 square miles, which is more representative of older master plans with large catchments and limited routing. The last three models (3, 4, and 5) used the original 62 sub-catchments in CUHP but included various levels of adjustment to the SWMM routing elements for calibration. The third model adjusted the conveyance slopes along the main channel of East Toll Gate Creek using outlet offsets to better represent the flatter channel slope with check and/or drop structures as shown in the HEC-RAS profile from the FHAD.

The fourth model included the adjusted channel slopes but also included more representative channel cross-sections from the FHAD HEC-RAS model than the 8-point cross-sections provided in the original model. The Manning's n value for the channel was calculated using Equation RO-10 from the USDCM and was based on the length-weighted slope of the reach and the hydraulic radius of the 10-year peak discharge from the HEC-RAS model. The calculated Manning's n values ranged from 0.037 to 0.048 and were actually less than the original master plan values for some cross-sections. The Manning's n values from the overbank areas were left consistent with the original master plan.

The fifth model was the same as the fourth model except that the Manning's n values for the cross-sections were increased, hopefully within a reasonable range of 0.035 to 0.075 to provide a better fit to the peak discharges given by the second model with only four sub-catchments. Table 2 summarizes the peak discharge results for all five models at four different locations in the catchment. Table 3 summarizes the Manning's n values used for all the test models.

Table 2. Comparison of Peak Discharge Results for Five ETGC Models.

| | (1) | | (2) | | (3) | | (4) | | (5) | |
|---|---|---|---|---|---|---|---|---|---|---|
| Design Point | Original MP Model | | 4 Sub-Catchment Model | | Calibration 1 Slope Adjusted | | Calibration 2 Eqn RO-10 Manning's n and Detailed XS | | Calibration 3 Manning's n | |
| | Peak Q (cfs) | Time to peak hr:mm | Peak Q (cfs) | Time to peak hr:mm | Peak Q (cfs) | Time to peak hr:mm | Peak Q (cfs) | Time to peak hr:mm | Peak Q (cfs) | Time to peak hr:mm |
| Upper ETGC (182) | **1,529** | 1:28 | **755** | 2:02 | **1,436** | 1:29 | **1,389** | 1:26 | **1,248** | 1:32 |
| Middle ETGC (172) | **4,697** | 1:05 | **4,741** | 1:04 | **4,649** | 1:04 | **4,733** | 1:04 | **4,723** | 1:02 |
| Lower ETGC (155) | **6,890** | 1:30 | **6,106** | 1:34 | **6,439** | 1:37 | **6,435** | 1:39 | **6,082** | 1:48 |
| ETGC Outfall | **6,809** | 1:36 | **6,040** | 1:41 | **6,380** | 1:45 | **6,419** | 1:43 | **6,057** | 1:54 |

As can be seen from the results in Table 2, the original master plan model results at the outfall were approximately 800 cfs greater than the results from the four major sub-catchments model, which we believe should be the baseline to keep results consistent with how CUHP was originally calibrated. However, the third round of calibration (Model 5) was able to bring the peak discharge results within 1% of the baseline at the outfall and at the Middle and Lower ETGC design points. Figure 9 shows the outfall hydrographs for Models 1, 2 and 5. The calibrated SWMM routing model was able to match the peak discharge at the outfall almost exactly but resulted in a slightly longer time to peak in the process. To achieve this, the Manning's n values (see Calibrated Model 5 in Table 3) had to be adjusted arbitrarily to achieve comparability with baseline peak discharges. This approach, although it can be made to work, does not provide consistency in how to best adjust Manning's n values in SWMM.

Table 3. Comparison of Manning's n used for Five ETGC Models.

| | Original Master Plan Model 1, 2 & 3 | | | Model 4 | Calibrated Model 5 | | |
|---|---|---|---|---|---|---|---|
| Reach XS | LB | RB | Chan'l | Chan'l | LB | RB | Chan'l |
| Hampden to Yale | 0.059 | 0.059 | 0.052 | 0.047 | 0.075 | 0.075 | 0.060 |
| Yale to Jewell | 0.050 | 0.050 | 0.040 | 0.043 | 0.075 | 0.075 | 0.060 |
| Jewell to Mississippi | 0.045 | 0.045 | 0.035 | 0.047 | 0.075 | 0.075 | 0.060 |
| Mississippi to Confl. w/ ETG Tributary | 0.059 | 0.059 | 0.052 | 0.048 | 0.060 | 0.060 | 0.035 |
| Confl. w/ ETG Trib. to Confl. w/ Columbia Ck. | 0.059 | 0.059 | 0.052 | 0.048 | 0.070 | 0.070 | 0.055 |
| Confl. w/ Columbia Ck. to Confl. w/ WTGC | 0.050 | 0.050 | 0.038 | 0.037 | 0.070 | 0.070 | 0.060 |

It can also be seen in Table 2 that the calibrated model was able to get closer to the peak discharge in the upper portion of ETGC, but is still considerably higher than what was produced by the four sub-catchment model. After further review, it was determined that the major subcatchment at the upstream end of the ETGC catchment was affected by the low imperviousness (approximately 6%). This low imperviousness resulted in a high Ct value and a low Cp value which had a significant effect on the peak discharge (less than 50% of the original MP model) and time to peak for the major sub-catchment (more than 30 minutes longer). However, in looking at the unit discharges at this location, the single major sub-catchment only produced 0.47 cfs/acre. The third round of calibration (Model 5) produced a combined unit discharge of 0.77 cfs/acre which is closer to expectations for an undeveloped watershed.

We typically expect around one cfs per acre for undeveloped lands with Type C-D soils in a catchment of around one square mile. The large 2.5 square mile catchment resulted in significantly lesser unit discharges than that. However, the results from the "calibrated" runs came closer to expectations for undeveloped land areas of this size. Regardless, it appears unlikely that adjusting SWMM channel parameters can get the numbers down to the large catchment discharges.

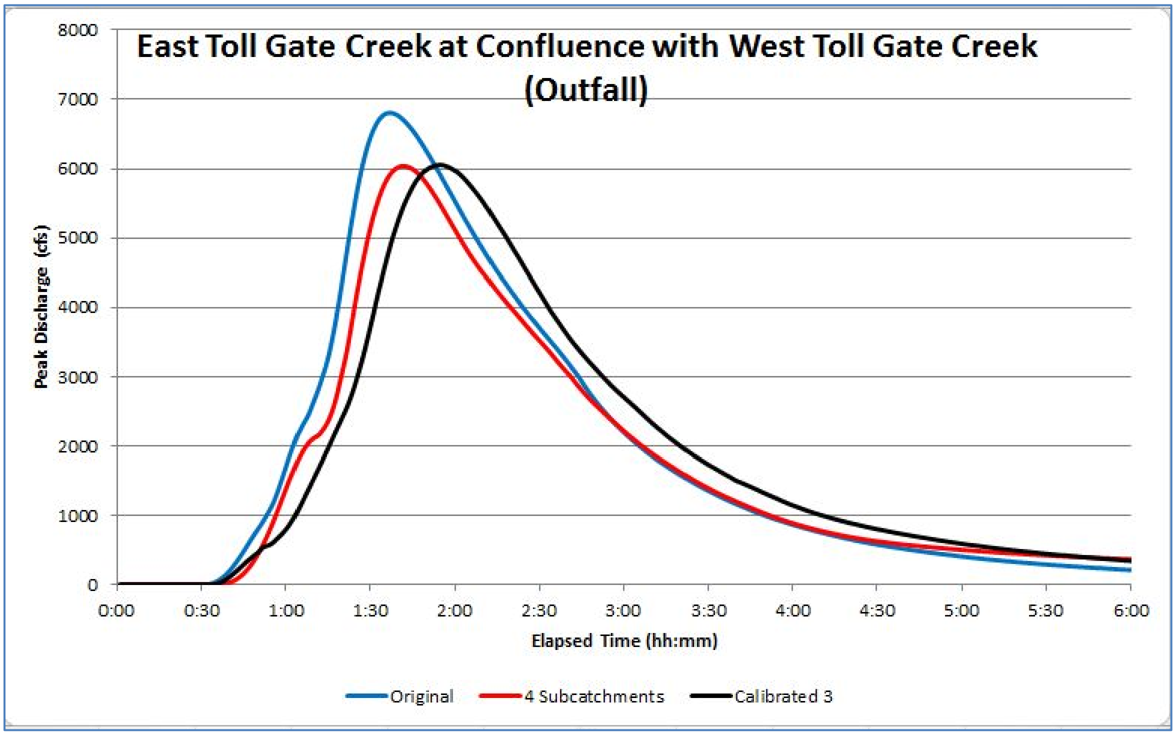


Figure 9. East Toll Gate Creek Outfall Hydrograph Comparisons

**Sand Creek**

The Sand Creek hydrology model from the master plan includes a study area of 13.68-square miles from Colfax to Yale Avenue, plus an additional 90.65-square miles of tributary area from Murphy Creek, Coal Creek and Senac Creek (see Figure 10). A total of 234 sub-catchments were included in the CUHP model, excluding Murphy Creek which was simply represented by an inflow hydrograph from a previous study. Of this total area, only the portion of Sand Creek within the MP study area (13.68-square miles) was evaluated as part of this test because that is the only portion that the HEC-RAS model covered in the FHAD.

The MP evaluated various rainfall depth-area adjustments for the watershed due to its large size. The goal was to determine what type of storm would produce the highest peak discharges, a storm covering the entire watershed with a low intensity, or a storm only covering part of the watershed with a higher intensity. The conclusion reached in the MP was to use a storm that covered the entire 104.33-square mile watershed. Initial attempts to re-create the CUHP results from the MP using the current version of CUHP (v1.4.3) failed because the rainfall depth reduction factors have been recently updated, resulting in less intense rainfall for large watersheds. Therefore, the analysis discussed below includes two sets of results, one using the 100-year rainfall distributions from the MP and the other using the rainfall distributions developed in CUHP v1.4.3. At the downstream end of the study, the MP reported a peak discharge of 19,600 cfs whereas the current version of CUHP along with SWMM routing only produced a peak discharge of 15,800 cfs. This analysis indicates that the low intensity storm covering the entire watershed may not be the storm that produces the peak discharge at the downstream study boundary, but that a more intense storm over a smaller area may produce the largest peak. However, it is not the purpose of this analysis to determine the critical storm.

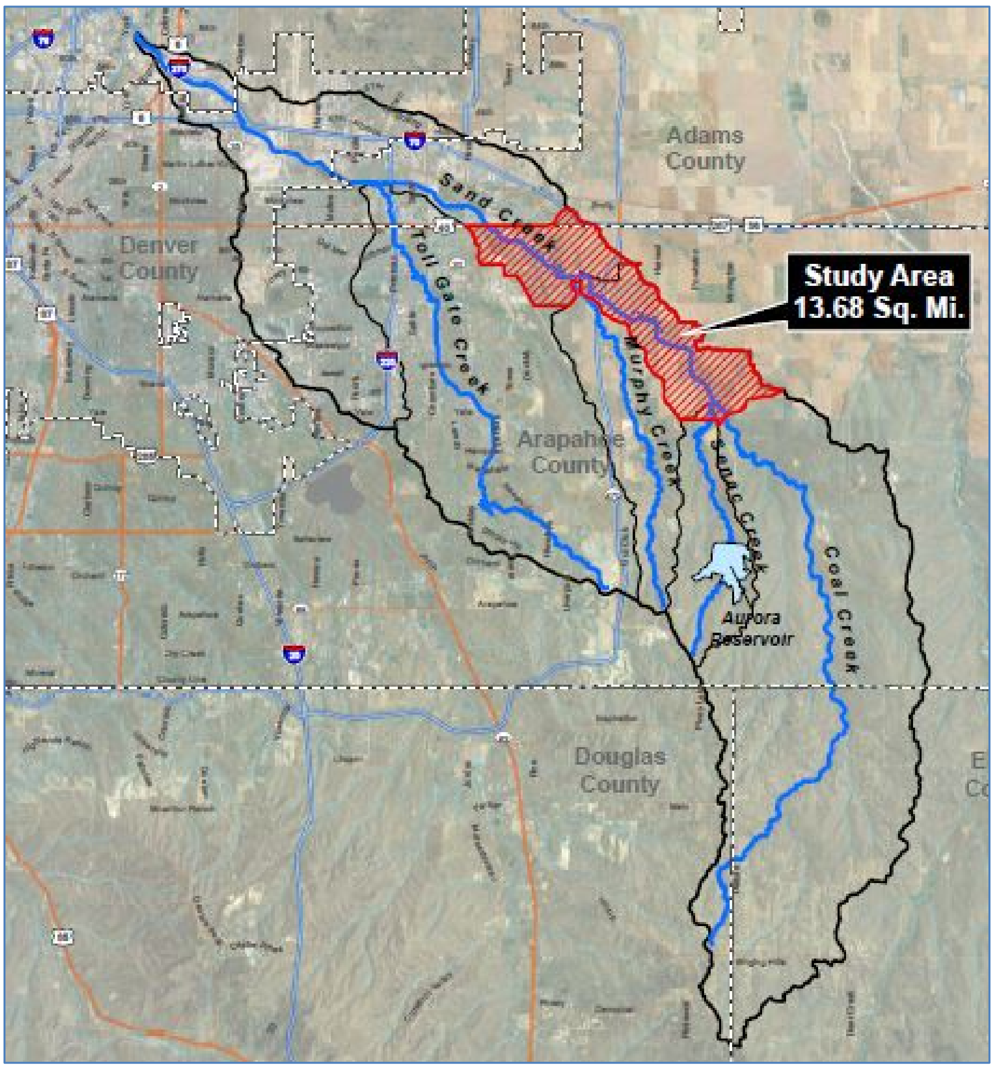


Figure 10. Catchment delineation for Sand Creek hydrologic model

Initially, the intent of this evaluation was to condense the highly discretized sub-catchments in the Sand Creek watershed to develop target peak discharges at various locations, similar to the approach used for Sanderson Gulch and East Toll Gate Creek. However, several factors complicated this approach including; the inflow hydrograph for Murphy Creek (12.60-square miles), the large size of the watershed, and the fact that the HEC-RAS models did not extend up into Senac Creek (9.59- square miles) or Coal Creek (68.46-square miles). Therefore, rather than develop idealized target peak discharges, it was decided to focus on adjusting the SWMM routing parameters and to compare the resulting peak discharges to the reported values in the MP. This decision was further supported by the fact that the MP report discussed how the channel cross-sections were determined using two-foot topographic contours and Manning's n values were increased by 25% in an attempt to be consistent with USDCM criteria.

The first SWMM routing parameter evaluated was longitudinal channel slope. The reach of Sand Creek within the study area had no discernable drop structures in the HEC-RAS channel profile. Channel slopes from the HEC-RAS model were compared to the slopes used in the SWMM model and were found to be within 0.02% of each other. Therefore, no adjustments were made to the longitudinal slope for the channels in the SWMM model.

Next, the SWMM channel cross-sections were replaced with representative irregular cross-sections from the HEC-RAS model. It should be noted that the original trapezoidal cross-sections in the SWMM model were actually quite detailed. Each channel had a unique bottom width with different side slopes for each side of the channel. It is apparent that the original trapezoidal channels were carefully measured from the topographic mapping as opposed to the more common approach of using a generic cross-section for all reaches.

The third SWMM channel routing parameter to be evaluated was Manning's n. The Manning's n values from the HEC-RAS model were used for the channel and overbank sections of the irregular cross-sections when they were input into SWMM. The original SWMM trapezoidal channels had a Manning's n value that appeared to be roughly the average of the channel and overbank values from the HEC-RAS model.

Figure 11 shows the modelled peak discharge profiles for Sand Creek from Colfax Avenue upstream to Yale Avenue. The two bottom peak discharge profiles reflect the rainfall distributions from CUHP v1.4.3 whereas the top two profiles reflect the rainfall distributions from the 2011 MP. The differences due to the HEC-RAS irregular cross-sections and the Manning's n values for the channel and overbank are negligible, as shown by the tight match between the peak discharge profiles for each rainfall distribution. This indicates that the Sand Creek SWMM routing model in the MP was well developed and included sufficient detail to accurately reflect the travel times through the watershed. Therefore, no further calibration of Manning's n values was required to get a better fit.

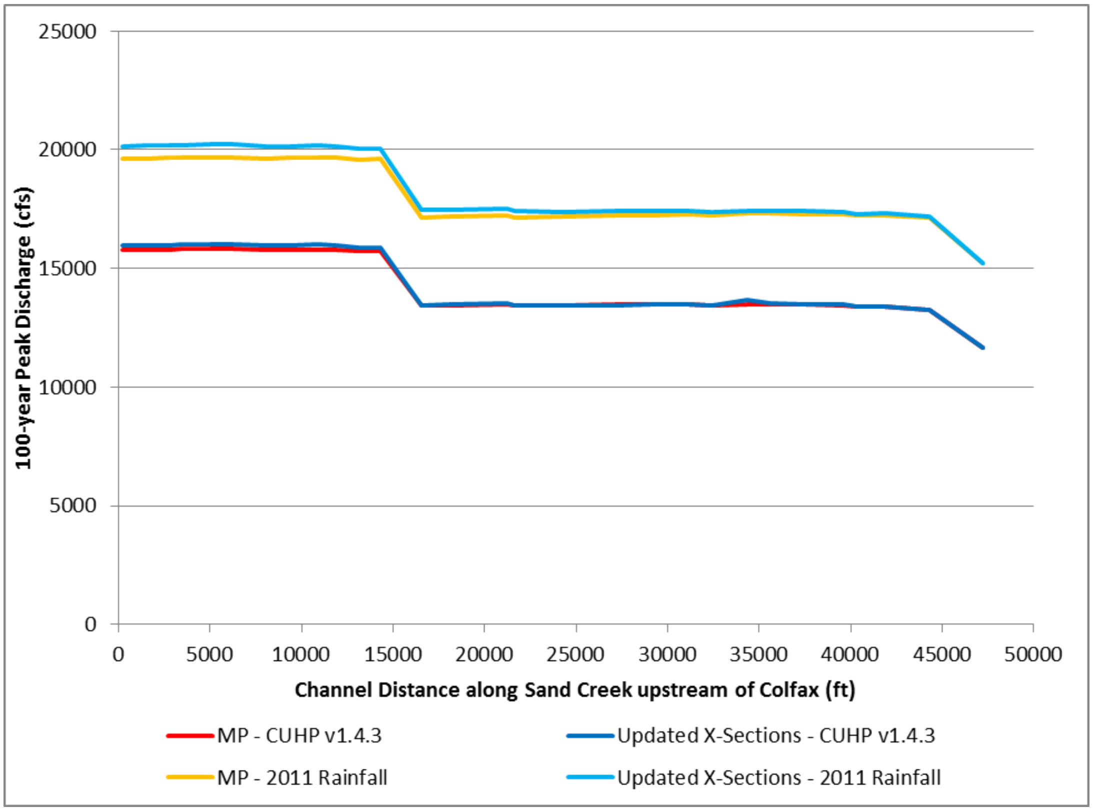


Figure 11. Comparison of 100-year peak discharge profiles for Sand Creek

**Observations Regarding differences between SWMM and CUHP Routing**

As was discussed near the beginning of this report, CUHP results reflect the surface and conveyance routing characteristics of the data from test catchments used to calibrate it. What this means is that all the nuances of runoff and routing within each catchment are reflected in the CUHP results. In other words, it accounts for all the street flow efficiencies and inefficiencies, the efficiencies of inlets, the multiples of bends and curves the water takes before it gets to the main conveyance and then all the bends and curves and other unmeasurable nuances in the major conveyance elements. On the other hand, the routing by SWMM is entirely dependent on the detail entered into it by the user. All SWMM conveyance elements are idealized mathematical representations of the runoff/routing process and it is left to the user to “calibrate” the model to give “realistic” results.

Unfortunately, calibration of SWMM lacks site specific data in sufficient numbers to make it “realistic”. The reliance is to calibrate it against CUHP, which has been calibrated using regional rainfall and runoff data. It is not expected that CUHP gives completely accurate results, something that is probably not possible in hydrology anyways. But, it does provide a target set of values that are representative of this region’s hydrology and are not subject to multiple guesses about which parameters in a model will give the most accurate results.

It is possible to adjust CUHP coefficient Cp for small sub-catchments when modelling large catchments. However, as was discussed earlier, this approach can result in very heavy data management and multiples of linked CUHP-SWMM runs, complicating the master planning studies. A preferred approach is to come up with generalized guidance on how to define the various routing elements within SWMM and then route the CUHP generated sub-catchment hydrographs through SWMM to better reflect the nuances of runoff routing through a system of conveyance elements. Patterns were observed in response to changes in various input parameters to define SWMM conveyance elements (channels in the case of the three previously master planned areas). It has become apparent that when setting up SWMM, it is best to:

1. Carefully estimate the representative effective longitudinal channel slopes instead of relying on the elevations at the two ends of each routing element. Often the longitudinal slope is controlled by natural or man-made drops, culverts, bridges, etc. and appropriate vertical offsets need to be incorporated at each junction to reflect the effective slope.
2. Define channel cross-sections to be representative of what is present in the field or what will be there in the future.
3. Use an appropriate Manning's n that is reflective of the many nuances in channel geometry and other flow controls along its reaches. Namely use ones recommend in the Urban Storm Drainage Criteria Manual (USDCM):
   a. For lined channels and pipes increase Manning's n by 25% over what would normally be used for the design
   b. For grass-lined, riprap-lined and natural channels use the higher range of the values for the appropriate type of channel reach as recommended by FHWA (see Appendix A).
   c. Whenever HEC-RAS sections are available from FHAD/MP Studies, the roughness coefficients from those studies can be used unless the values obtained using recommendation in item 3.b above are higher.

The USDCM currently recommends the use of Equation RO-10 (i.e., Jarret's USGS equation for steep gradient streams) to determine Manning's n for grass-lined, riprap-lined and natural channels. However, the use of this equation can be problematic for planning purposes because it is dependent on the hydraulic radius of the channel which is determined by assuming a peak flow equal to one half of the estimated hydrograph peak flow. Unfortunately, the peak flows are most often not known when setting up a master plan model which results in an iterative process to determine the appropriate Manning's n values for the channel reaches. Calculation of the hydraulic radius for irregular channels can further complicate the process. Therefore, it is recommended that the FHWA table in Appendix A be used to determine Manning's n values for master planning, FHAD and channel rehabilitation models.

However, because HEC-RAS models were available, and because the hydraulic radius is easily obtained from the HEC-RAS model for the 10-year peak discharge, the effects of Equation RO-10 on the Manning's n values were tested for the three master plans provided for this study. In addition, the effects of roughness values were tested by increasing RO-10 values by 15% and

25%. The results for Sand Creek did not vary much as a result of changing Manning's n values, primarily because the values calculated from Equation RO-10 were only slightly lower than the values in the MP and FHAD.

The results for Toll Gate Creek catchment are summarized in Table 4 and for Sanderson Gulch in Table 5. As can be seen for Toll Gate Creek in Table 4, the incremental changes in the peak discharges are very small after the model has been adjusted to have representative natural cross-sections and longitudinal slopes (i.e. column 3). Increasing Manning's n by 15% and 25% showed little change even though the 15% increase did bring the numbers more in line with the baseline condition in column 2.

On the other hand, similar changes to Manning's n for Sanderson Gulch had greater impact on peak discharges. Use of Eq-10 roughness values brought the mid-point peaks very close to the base-line condition, but understated them at the catchment's outfall. However the results are still within the expected confidence limits for this type of analysis and much better than the original master plan model. Parameter testing on Toll Gate Creek and Sanderson Gulch did not extend into the tributaries and were limited to the major drainageways only.

Table 4. Toll Gate Creek's peak discharges from varying Eqn. RO-10 values and other runs.

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|
| | **Original MP Model** | **4 Sub-Catchm't Model** | **Slope Adjust & HEC-RAS x-sect** | **Add Use of Eqn RO-10 Manning's n** | **Use Eqn RO-10 + 15%** | **Use Eqn RO-10 + 25%** | **Ideal Calibr'n of Manning's n** |
| **Design Point** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** |
| | % Diff from 2 | | % Diff from col. 2 | % Diff from col. 2 | % Diff from col. 2 | % Diff from col. 2 | % Diff from col. 2 |
| **Upper ETGC (182)** | **1,529** | **755** | **1,436** | **1,389** | **1,325** | **1,436** | **1,248** |
| | 103% | | 90% | 84% | 75% | 90% | 65% |
| **Middle ETGC (172)** | **4,697** | **4,741** | **4,649** | **4,733** | **4,654** | **4,649** | **4,723** |
| | -1% | | -2% | 0% | -2% | -2% | 0% |
| **Lower ETGC (155)** | **6,890** | **6,106** | **6,439** | **6,435** | **6,064** | **6,439** | **6,082** |
| | 13% | | 5% | 5% | -1% | 5% | 0% |
| **ETGC Outfall** | **6,809** | **6,040** | **6,380** | **6,419** | **6,044** | **6,380** | **6,057** |
| | 13% | | 6% | 6% | 0% | 6% | 0% |

Table 5. Sanderson Gulch's peak discharges from varying Eqn. RO-10 values and other runs.

| | 1 | 2 | 3 | 4 | 5 | 6 | 7 |
|---|---|---|---|---|---|---|---|
| | **Original MP Model** | **2 Sub-Catchm't Model** | **Slope Adjust & HEC-RAS x-Sect** | **Add Use of Eqn RO-10 Manning's n** | **Use Eqn RO-10 + 15%** | **Add Use of Eqn RO-10 + 25%** | **Ideal Calibr'n of Manning's n** |
| **Design Point** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** | **Peak Q (cfs)** |
| | % Diff from 2 | | % Diff from 2 | % Diff from 2 | % Diff from 2 | % Diff from 2 | % Diff from 2 |
| **Mid-point** | **6,909** | **4859** | **5,810** | **4,696** | **4,379** | **n/a** | **5,810** |
| | 42% | | 20% | -3% | -10% | n/a | 20% |
| **Outfall** | **11,769** | **8,528** | **8,620** | **6,717** | **6,061** | **n/a** | **8,620** |
| | 38% | | 1% | -21% | -29% | n/a | 1% |

Although it is possible to achieve almost perfect comparison with the baseline as illustrated in column 7 by manipulating the roughness coefficient values in combination with the adjusted slopes and representative cross-sections, one has to wonder how accurate the baseline is itself. It is somewhat representative of what CUHP with 2.5-square mile catchments would produce after routing the results by SWMM and has its own band of confidence that could be on the order of 25%. In other words, pursuit of "perfection" is just that which adds only to the time spent on hydrology studies without commensurate improvements in accuracy. Because of this, it is suggested that after the three steps listed above have been carefully implemented when using linked CUHP-SWMM models with dense discretization, UDFCD may want to consider the results be accepted for use in the master planning and FHAD studies unless it is committed to meeting other target peak discharges from earlier master planning efforts or local data.

## OBSERVATIONS AND RECOMMENDATIONS

The effects of increased unit discharges due to discretizing large catchments into small ones and then combining the resultant CUHP hydrographs using SWMM can be addressed in a couple of ways. One is to adjust the CUHP parameter Cp. The other is to develop guidance on how to develop appropriate SWMM parameter to provide more realistic hydraulic routing through the conveyance network. Although the former can be made to achieve consistent results, the user may need to perform a series of adjusted runs in order to get the appropriate peak discharges for different locations within a large planning catchment. On the other hand, the latter approach is consistent at all locations in the catchment. After testing if SWMM routing could be adjusted to produce reasonable final results for master planning and FHAD studies, it was decided to recommend that it be used. However, it will require discipline in how that is done and the following protocol is recommended:

1. Carefully estimate the representative effective longitudinal channel slopes instead of relying on the elevations at the two ends of each routing element. Often the longitudinal slope is

controlled by natural or man-made drops, culverts, bridges, etc. and appropriate vertical offsets need to be incorporated at each junction to reflect the effective slope.

2. Define channel cross-sections to be representative of what is present in the field, or what will be there in the future
3. Use an appropriate Manning's n that is reflective of the many nuances in channel geometry and other flow controls along its reaches. Namely use ones recommend in the Urban Storm Drainage Criteria Manual (USDCM):
   a. For lined channels and pipes increase Manning's n by 25% over what would normally be used for the design
   b. For grass-lined, riprap-lined and natural channels use the higher range of the values for the appropriate type of channel reach as recommended by FHWA (see Appendix A).
   c. Whenever HEC-RAS sections are available for FHAD/MP Studies, the roughness coefficients for the main channel and overbanks from those studies can be used unless the values obtained using recommendation in item 3.b above are higher

The above procedure in setting up SWMM is to achieve consistency with CUHP for larger catchments. It is not the complete answer when trying to duplicate results reported in older master planning hydrology studies. If UDFCD wants to calibrate the new hydrology results to approximately match the old ones, it may require more steps than recommended above such as making sure the same rainfall distributions are used. We recommend that the above steps be applied first and if they are not adequate to achieve the desired results and need further calibration, adjustments to CUHP parameter Cp be investigated on a case-by-case basis.

## Appendix A – Manning’s n for Natural Streams and Floodplains

(Ref.: Federal Highway Administration, http://www.fhwa.dot.gov/engineering/hydraulics)

| | |
|---|---|
| **MINOR STREAMS (top width at flood stage < 30 m)** | |
| **Streams on Plain** | |
| 1. Clean, straight, full stage, no rifts or deep pools | 0.025–0.033 |
| 2. Same as above, but more stones and weeds | 0.030–0.040 |
| 3. Clean, winding, some pools and shoals | 0.033–0.045 |
| 4. Same as above, but some weeds and stones | 0.035–0.050 |
| 5. Same as above, lower stages, more ineffective slopes and sections | 0.040–0.055 |
| 6. Same as 4, but more stones | 0.045–0.060 |
| 7. Sluggish reaches, weedy, deep pools | 0.050–0.080 |
| 8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush | 0.075–0.150 |
| **Mountain Streams, no Vegetation in Channel, Banks Usually Steep, Trees and Brush Along Banks Submerged at High Stages** | |
| 1. Bottom: gavels, cobbles and few boulders | 0.030–0.050 |
| 2. Bottom: cobbles with large boulders | 0.040–0.070 |
| **FLOODPLAINS** | |
| **Pasture, No Brush** | |
| 1. Short Grass | 0.025–0.035 |
| 2. High Grass | 0.030–0.050 |
| **Cultivated Areas** | |
| 1. No Crop | 0.020–0.040 |
| 2. Mature Row Crops | 0.025–0.045 |
| 3. Mature Field Crops | 0.030–0.050 |
| **Brush** | |
| 1. Scattered brush, heavy weeds | 0.035–0.070 |
| 2. Light brush and trees in winter | 0.035–0.060 |
| 3. Light brush and trees in summer | 0.040–0.080 |
| 4. Medium to dense brush in winter | 0.045–0.110 |
| 5. Medium to dense brush in summer | 0.070–0.160 |
| **Trees** | |
| 1. Dense willows, summer, straight | 0.110–0.200 |
| 2. Cleared land with tree stumps, no sprouts | 0.030–0.050 |
| 3. Same as above, but with heavy growth of sprouts | 0.050–0.080 |
| 4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches | 0.080–0.120 |
| 5. Same as above, but with flood stage reaching branches | 0.100–0.160 |
| **MAJOR STREAMS (Top width at flood stage > 30 m)** | |
| The n value is less than that for minor streams of similar description, because banks offer less effective resistance. | |
| Regular section with no boulders or brush | 0.025–0.060 |
| Irregular and rough section | 0.035–0.100 |