Urban Runoff in the U.S. Southwest –
The Importance of Impervious Surfaces for Small Storm Hydrology

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Abstract

Impervious surfaces have long been recognized as one of the critical factors influencing the rainfall-runoff relationship in urban areas. Published guidance on how to treat urban imperviousness in hydrologic simulations varies, and few studies exist for the southwestern United States. In recent years, small storm hydrology has become increasingly important to support water quality evaluations. Methods developed for large, infrequent storms are often applied to simulate runoff from smaller rain events, with questionable results. To test the impact of imperviousness on small storm runoff, impervious cover in a 1.5 km^2 urban basin in central New Mexico was mapped and differentiated into directly connected impervious areas (DCIA) and unconnected impervious areas (UIA) that are separated from the drainage system by a pervious buffer. Measured runoff from 25 small magnitude storms was compared to model scenarios with varying treatment of imperviousness. Simulation results revealed that during small rain events, all runoff originated from DCIA, while UIA contributed no flow. For this particular basin with 28% DCIA and 16% UIA, losses from UIA approached a constant value of approximately 4 mm. Findings were applied to a 142 km^2 watershed to assess model performance for larger storms. The impact of losses from UIA on the runoff response decreased with increasing storm magnitude. This study presents a modeling approach for DCIA and UIA that increased accuracy for small runoff events in urban areas without negatively affecting model performance for larger storms. Use of this method can improve model predictions, particularly when hydrologic simulations built with flood control in mind are used to assess the impact of more frequent storm events.

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Introduction

Impervious surfaces have long been recognized as one of the critical factors influencing the rainfall-runoff relationship in urban areas (Shuster et al., 2005). Urban imperviousness not only causes higher peak discharges and runoff volumes from a given storm, it also leads to an increased frequency of runoff events (Ladson et al., 2006). Negative impacts on water quality and stream health have been well documented in the published literature (Arnold and Gibbons, 1996; Schueler et al., 2008).

Impervious surfaces can be divided into two categories: directly connected and unconnected (Boyd et al., 1993). Directly connected impervious areas (DCIA) comprise all surfaces with a direct connection to the drainage system. Impervious areas that drain onto pervious surfaces, e.g. rooftop areas that drain onto landscaping, are considered unconnected impervious areas (UIA). This distinction is important during small storm events because some or all of the runoff from UIA may spread over pervious surfaces and infiltrate before it reaches the drainage system. Lee and Heaney (2003) studied a residential area in Miami and found that, during a 52-year period, DCIA alone contributed more than 70 % of total runoff.

Lumped parameter models commonly used in hydrologic analyses require the delineation of subbasins with relatively uniform land use characteristics (Cronshey et al., 1986). This poses a challenge in urban areas, where pervious and impervious surfaces exist in close proximity. Two general strategies exist for modeling imperviousness:

1. Two separate hydrographs are calculated for the pervious and impervious portions of each subbasin. The hydrographs are then added to obtain the total subbasin runoff. This technique has been referred to as the “split hydrograph method” (Anderson et al., 2005). It is often assumed that impervious areas are not subject to losses, and all contributing precipitation is converted to runoff (USACE, 2000).
2. A second strategy incorporates imperviousness into a composite loss calculation for each subbasin. The most prominent example for this approach is the curve number (CN) method (USDA, 1954), one of the most widely used empirical models for estimating direct runoff from a storm event (Hagen, 1995).

All hydrologic models are simplified representations of complex real-world processes. Models are sensitive to precipitation intensity, distribution, depth, and duration (Obled et al., 1994; Arnaud et al., 2011).

In the past, hydrologic analyses were often used solely for flood control and infrastructure design purposes. Consequently, methods for simulating the rainfall-runoff relationship were developed and calibrated for infrequent, large magnitude storms. Following the 1987 amendment to the Clean Water Act of 1972 and subsequent regulations for stromwater discharges by the U.S. Environmental Protection Agency, all 50 U.S. states have adopted standards for stormwater discharges (USEPA, 2016). Since many of the standards target frequent, small magnitude storm events, there has been an increasing need to quantify runoff from such storms.

Pit (1999) and Garen and Moore (2005) found that procedures developed for large magnitude design storms such as the curve number method have frequently been used to simulate runoff from small storm events, with questionable results. Poor performance of the CN method for small storms in urban basins has been linked to the handling of imperviousness in the model (Cronshey et al, 1986; Golding, 1997; NJDEP, 2004).

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The objective of this study was to find a method of representing impervious surfaces in HEC-HMS (USACE, 2013a) that would produce acceptable simulation results both for small, frequent storm events, as well as larger storms with higher recurrence intervals. HEC-HMS was selected because it is the recommended program for hydrologic analyses conducted in the study area (NMOSE, 2008; CoRR, 2009).

The study approach:

- Monitor precipitation and runoff from a small urban catchment in central New Mexico. A study area of small size allows accurate measurement of hydrologic input parameters such as precipitation and land use, thus reducing model uncertainty;
- Quantify impervious coverage in the study area, and assess which portion of the impervious surfaces are directly connected to the drainage system;
- Construct and calibrate a hydrologic model of the catchment using measured rainfall-runoff data;
- Assess how representation of impervious surfaces affects model results from 25 measured storm events of small magnitude; specifically, test model scenarios which include impervious areas in composite loss calculations, as opposed to simulations that treat pervious and impervious areas separately for the same catchment;
- Apply findings to a large watershed and assess model performance for 10 measured storms of varying magnitudes and durations.

Study Areas

Two watersheds were examined for this study, both located in Sandoval County, New Mexico. Study area A encompasses a 1.5 km^2 (150 ha) urban drainage basin located in the city of Rio Rancho. Land use in study area A (Fig. 1) is predominantly single family residential with an average density of 12 dwelling units per hectare. A commercial tract (11 hectares) with retail stores and restaurants is located at the south-east corner of the study area. In the upper part of the catchment, 12 hectares of land are still undeveloped. Two arterial roads form the eastern and southern boundary of the basin. Runoff drains to a trapezoidal concrete channel at the outlet of the watershed. Stormwater from approximately 88 hectares is captured by a detention pond, and is then conveyed to the channel via a storm drain. Runoff from the remaining area follows paved residential streets until intersecting one of the arterials, where it is captured by curb inlets and flows through storm drain pipes to the concrete channel. The catchment slopes from north-west to south-east with an average grade of 2.5%.

Study area B is a 142 km^2, partially urbanized watershed immediately to the north of area A. Approximately 20% is urbanized, predominantly in the lower and central part of the catchment (Fig. 2), and contains 48 stormwater detention ponds (Fig. S1). The rest of the basin remains largely in its natural state characterized by sandy, erosive soils, and semi-desert shrub and grasslands (Dortignac, 1956). Runoff drains through a network of natural channels called arroyos. For the last 3 km, stormwater is conveyed to the Rio Grande in a rectangular concrete channel. The watershed was selected for this study because rainfall and runoff data have been collected for about a decade, including several larger magnitude storm events. Average annual rainfall equals 250 mm, with values ranging from 100 to 400 mm (NOAA, 2016a).

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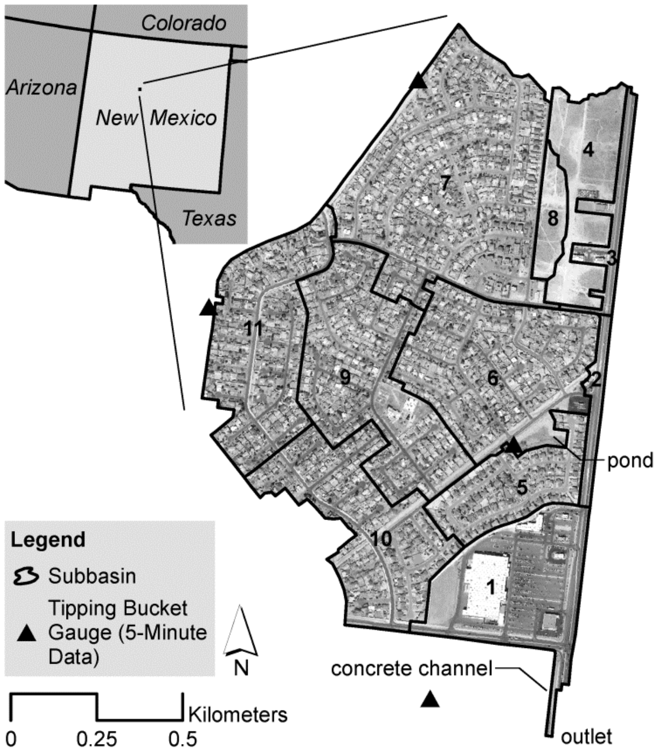

Fig. 1. Map of study area A with rain gauge locations.

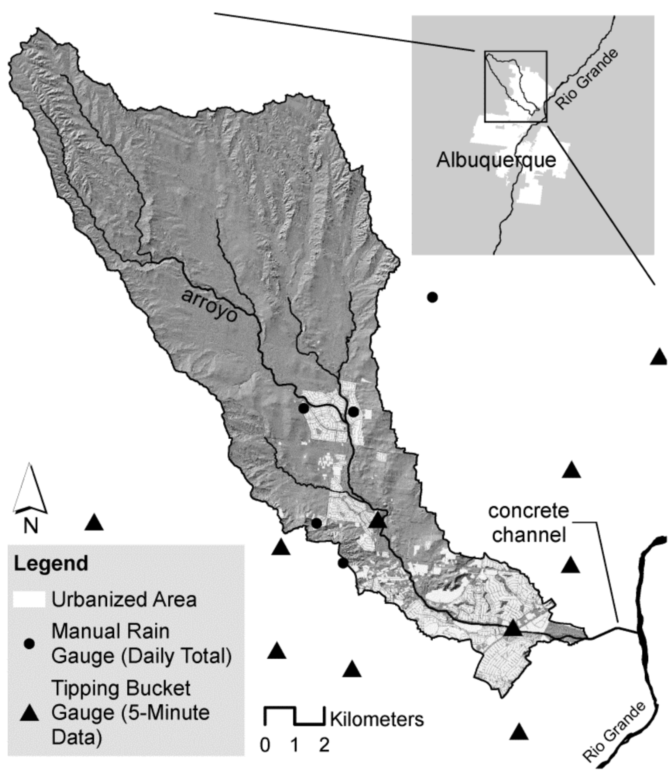

Fig. 2. Map of study area B with rain gauge locations.

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Methods

Observed Data

Rainfall in study area A was measured using four tipping bucket rain gauges (Onset Computer Corp., Bourne, MA, and Sutron Corp. Sterling, VA) with a resolution of 0.25 mm (0.01 in). During a 17-month period (April 2015 to September 2016), 25 storm events that produced runoff were observed (Table S1). Fig. 3 compares the measured storms to intensity-duration-frequency curves (IDF, see gray lines in Fig. 3) for study area A (NOAA, 2016b). Most storms were short, with durations ranging from 15 minutes to six hours. Recurrence intervals (RI) for 20 storms were less than 1 year (Fig. 3, dot). Five storms (solid) had recurrence intervals between 1-2 years, and only one 10-year storm (dash) of short duration was observed. Fig. 3 displays data from the rain gauge with the highest precipitation reading for each storm event, and does not represent basin average rainfall values.

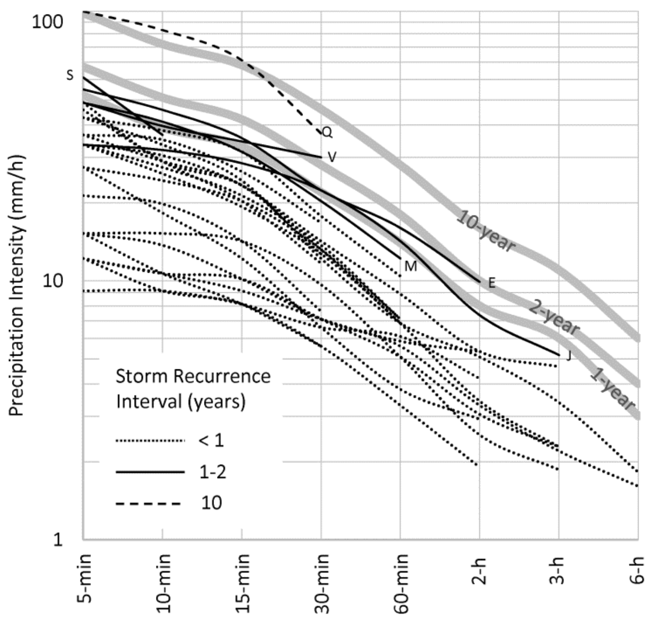

Fig. 3. Intensity-duration-frequency curves (grey) for study area A and observed storms (black).

Rainfall and runoff data from ten storm events that occurred between October 2008 and August 2014 were available for study area B (Table S2). Precipitation data were obtained from three sources: tipping bucket rain gauges, radar derived rainfall estimates (NOAA, 2016c), and storm total rainfall depths measured by volunteer weather observers. Fig. 4 shows a comparison of measured rainfall and IDF curves (NOAA, 2016b) from the location with the highest precipitation record for each storm event. Elements of two storms exceeded the 100-year RI (Fig. 4, short dash); portions of four storms fell between the 25- and 50-year RI (solid); one storm (dash-dot) approached the 10-year RI, two storm events fell close to the annual (1-year) storm (dot), and one storm was smaller than the annual storm (long dash). Spatial extent of storms in area B varied considerably; Figs. S2-S11 show maps of cumulative precipitation depths for each storm event. It is important to note that storm data presented in Fig. 4 does not represent basin average rainfall values. Moreover, none of the storms were close in magnitude to the 100-year 24-hour design storm, although the intensity of two storms exceeded the 100-year frequency for part of the storm durations in some portions of area B.

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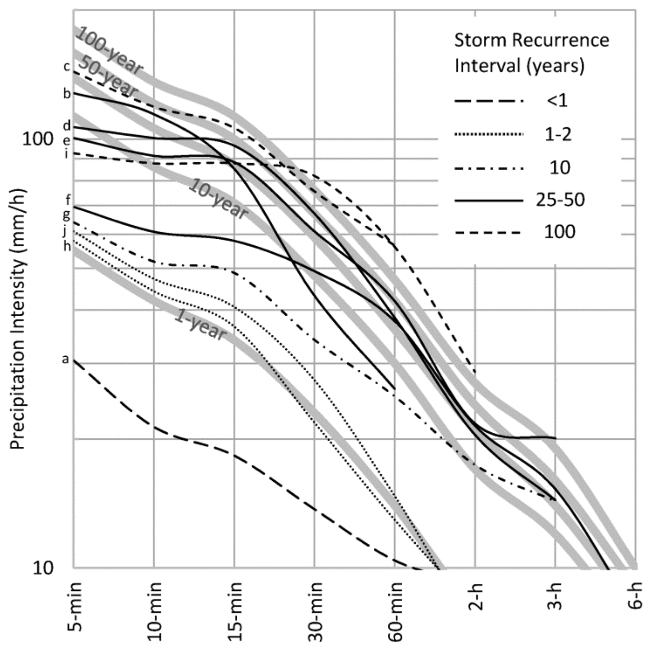

Fig. 4. Intensity-duration-frequency curves (grey) for study area B and observed storms (black).

Discharge at the outlet of study area A was measured using time-lapse photography similar to the setup proposed by Royem et al. (2012). A time lapse camera (Stealth Cam G-series, GSM, Grand Prairie, TX) with infrared flash was mounted on an existing channel crossing structure. Enclosed in a lockable steel housing, the camera points upstream to a staff gauge attached to the channel side wall (Fig. 5). The camera was set up in time lapse mode and configured to take one photo every five minutes. During flow events, depth readings from the staff gauge were recorded manually for every image and entered into a spreadsheet.

Since the staff gauge is attached to the side slope of a trapezoidal channel, flow depth was calculated based on the slope angle. Flow depth values for each five-minute interval were used to estimate discharge based on a theoretical rating curve developed in HEC-RAS version 4.1 (USACE, 2010). Fig. 5 shows examples of time-lapse imagery from a storm event on August 6, 2016. Peak flow occurred between 16:40 and 16:45 and left a distinct debris line (dashed black line, Fig. 5) on the side of the channel. The debris line is approximately 4 cm (vertical distance) higher than the water surface elevation apparent in the image and was used to estimate peak discharge. All other points on the hydrograph are based on water surface elevation from the corresponding photo. The same approach was applied to all other storms: in cases where a debris line was present and higher than the maximum water surface elevation visible in the photo, peak discharge was calculated based on the debris line.

Runoff from study area B was measured in the rectangular concrete channel near the watershed outlet using a pressure transducer (Level Troll 500, In-Situ, Fort Collins, CO). Similar to study area A, flow depth was recorded in 5-minute intervals and converted to discharge using a theoretical rating curve developed in HEC-RAS (USACE, 2010).

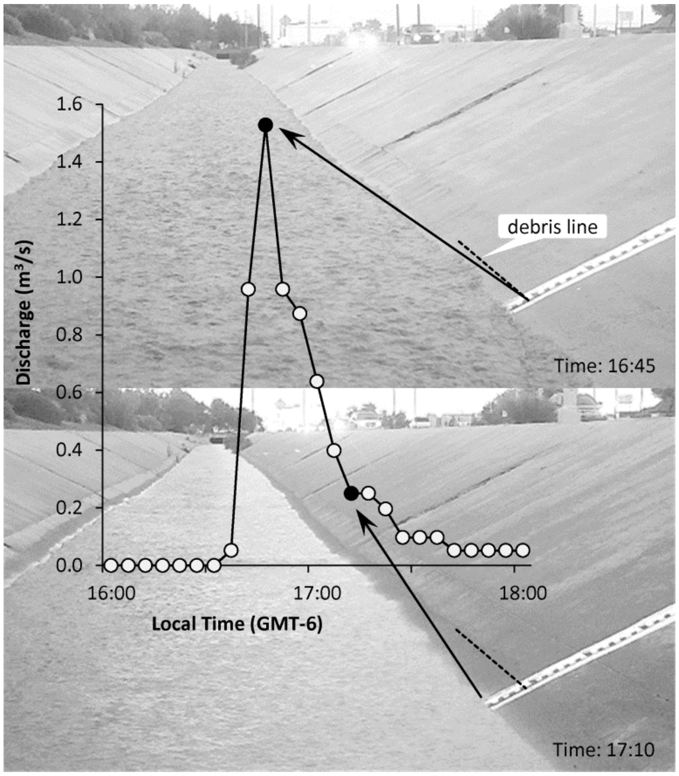

Fig. 5. Examples of time lapse imagery for the storm of August 6, 2016, and corresponding points on the hydrograph (images by author).

Hydrologic Models

HEC-HMS was selected for this study, as it is the recommended program for hydrologic analyses conducted in the study area (NMOSE, 2008; CoRR, 2009). Hydrologic models for both study areas were built in HEC-HMS version 4.0 (USACE, 2013a). The model for study area A includes 11 subbasins and one pond (Fig 1). The much larger study area B was divided into 143 subbasins connected by 133 routing reaches, and has 48 ponds (Fig. S1). Initial subbasin delineation was accomplished using HEC Geo-HMS software (USACE, 2013b) based on a digital elevation model created from LiDAR data with a vertical resolution of 0.6 m. Boundaries were modified to accommodate desired analysis points (e.g. ponds, tributary confluences, road crossings) and achieve basins with relatively uniform land use characteristics. Questionable boundaries were verified in the field.

Rainfall Input

Incremental (5-minute) point precipitation data were converted to rainfall grids with the inverse distance square weighted average interpolation method using the Gageinterp program (USACE, 1999). A 10-m grid was used for study area A, while a 500-m grid was applied to the much larger study area B. Only rain gauge data were used to for study area A. In study area B, gauge data were augmented with radar derived rainfall estimates (NEXRAD Level-III DTA/172, NOAA, 2016c) for three out of ten storms. Based on the areal extent of each storm, points were selected strategically to fill in gaps in the rain gauge coverage. At each point, radar estimates were converted to 5-minute time series of incremental precipitation and bias-corrected by multiplying each time step with a bias adjustment factor. The bias adjustment factor B for a

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given storm event was calculated as the median ratio of rain gauge and NEXRAD values for each rain gauge location:

$$B = Median\left(\frac{G_i}{R_i}\right);\ \ i = 1,2,\dots,n \quad (1)$$

where G_i is the rain gauge measured cumulative storm precipitation at location i, R_i is the radar estimated cumulative storm precipitation at location i, and n is the number of locations with rain gauge and radar data.

The bias adjustment method (Eq. 1) is similar to the procedure described by Bradley et al. (2002) for hourly data. The median ratio was chosen instead of the arithmetic average because it is less sensitive to the presence of outliers.

Loss Methodology

The curve number (CN) method, first published in 1954 by the Soil Conservation Service (USDA, 1954), is one of the most widely used empirical models for estimating direct runoff from a storm event. The CN method and underlying equations have been described in detail in the published literature (Ponce and Hawkins, 1996; USDA, 2004; Mishra et al., 2012). Originally developed based on data for agricultural catchments in the eastern half of the United States (Fennessey et al., 2001), use of the CN method has been extended to urban areas (Cronshey et al., 1986). Hagen (1995) estimated that 60% of hydrologic studies for urban watersheds in the United States used the CN method. Examples of more recent publications illustrate that the method is used for drainage analyses worldwide, including China (Fan et al., 2013), India (Mishra and Kansal, 2014), Brazil (Oliveira et al., 2016), Poland (Banasik et al., 2014) and Italy (D'Asaro and Grillone, 2012). Although originally not intended as an infiltration equation (Chen, 1982), the CN method has been widely used to simulate infiltration (Mishra and Singh, 2004; Zhou et. al, 2015), and has been incorporated as a loss subroutine into many hydrologic modeling programs including Win TR-20 (USDA, 2015), HEC-1 (USACE, 1998), HEC-HMS (USACE, 2013a), FLO-2D (2009), and SWMM (Rossman, 2015). The SCS curve number loss methodology in HEC-HMS allows users to input three parameters:

- ***Initial abstraction*** can be user specified, but is by default calculated as 20% of the potential retention, which in turn is calculated from the curve number. In this study, the default was used, and no values for initial abstraction were specified.
- ***Composite curve numbers*** represent all soil and cover types within a subbasin; several guidelines recommend including DCIA in the composite curve number (Cronshey et al., 1986; USACE, 2013a).
- ***Percent impervious*** **–** instead of including imperviousness in the composite CN, the user can specify the percentage of impervious area for each subbasin separately; if this option is selected, all precipitation that falls on the impervious portion of the subbasin becomes direct runoff (USACE, 2013a).

To estimate loss parameters, soil and cover types for study area A were evaluated based on several data sources. Land use, particularly impervious surfaces, were mapped in ArcGIS (2016) based on spatial data obtained from the City of Rio Rancho GIS department, along with aerial orthoimagery. Fig. 6 demonstrates the contrast between pervious (white) and total impervious areas (TIA, black) in study area A. Pervious cover include residential yards, desert landscaping, undeveloped land, and public open space. In Fig. 7, only directly connected impervious areas

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are shown in black. DCIA comprises paved roads and parking, residential driveways, and those sidewalks directly adjacent to roadways without a pervious buffer. Rooftop areas on commercial sites that drain directly onto impervious parking are also considered directly connected. Field investigation revealed that residential roofs typically drain onto pervious landscaping, and are thus not considered directly connected. The only exception is that portion of the roof that drains onto the driveway, often a portion of the garage roof. It was estimated that five percent of residential roofs are directly connected; buildings are therefore shown as light gray in Fig. 7. Fig. 8 shows types of imperviousness and their respective percentages of overall impervious surface in the catchment. DCIA is shown in black, UIA is indicated in white. Study area A comprises 44 % total imperviousness; DCIA accounts for 63% of total imperviousness (or 28% of the catchment area), UIA constitutes 37% of total imperviousness (or 16% of the basin). Table S3 lists percentages for pervious and impervious areas by subbasin for study area A.

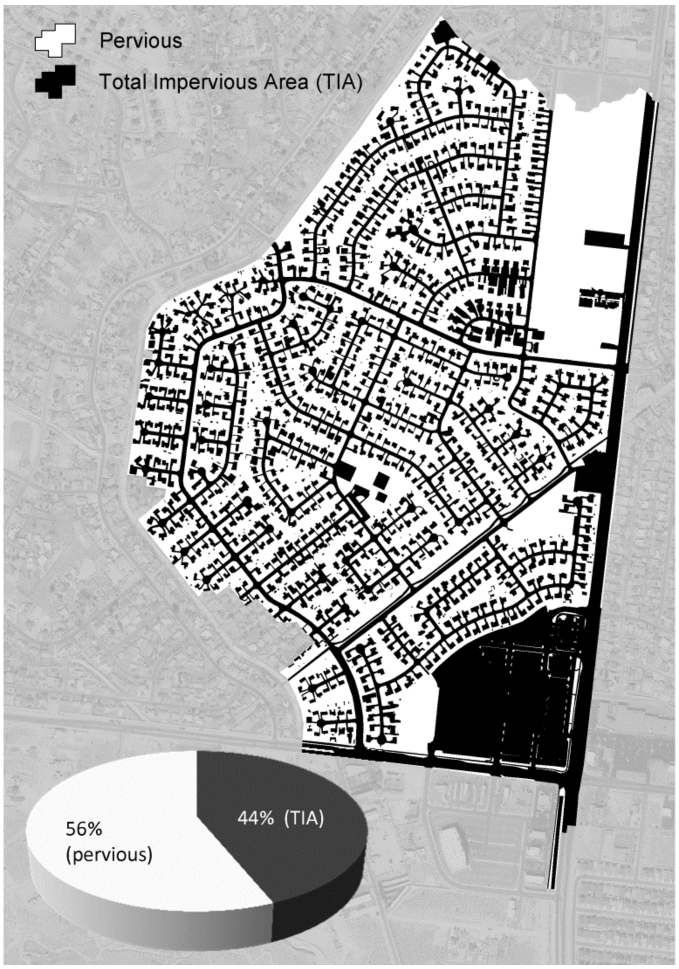

Fig. 6. Total impervious coverage (black) vs. pervious areas (white) in study area A.

The same approach was adopted for study area B. DCIA was mapped based on available GIS data sources and verified using aerial orthoimagery. Major contributors to DCIA – roads, and parking lots – received special attention. Driveways were quantified based on the number of developed, residential lots per subbasin, and the average residential driveway size as determined in study area A. Pervious cover was classified using soils data (USDA, 2016), land use information obtained from the City of Rio Rancho, and aerial orthoimagery. 5.1% of study area

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B is covered by impervious surfaces, DCIA accounts for 67 % of total imperviousness (or 3.4% of the watershed area). Table S4 lists pervious and imperious cover types for study area B.

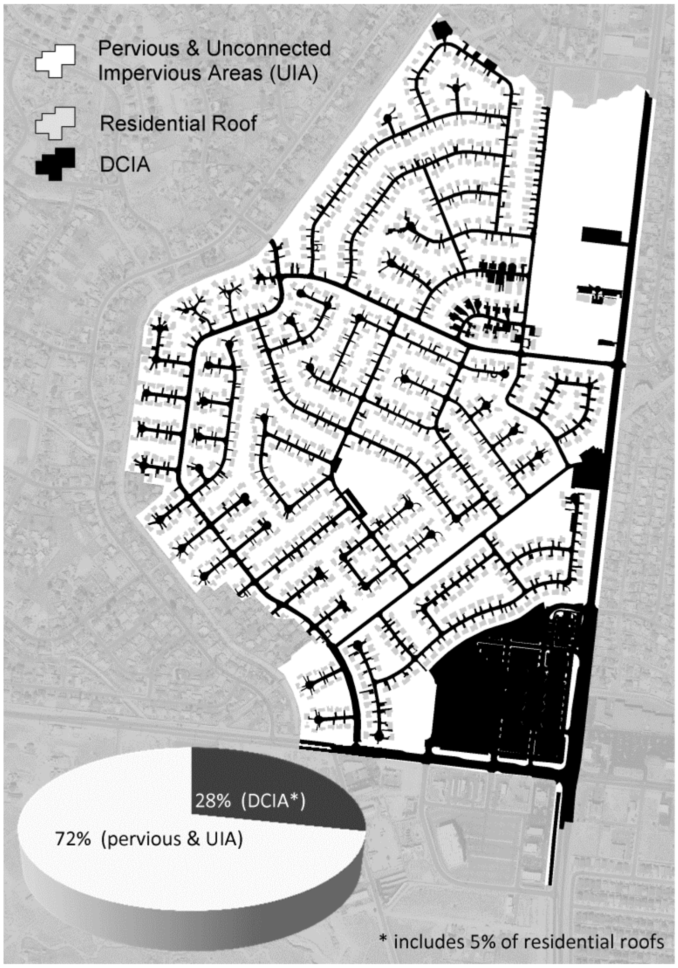

Fig. 7. Directly connected impervious areas (black) vs. pervious and unconnected impervious areas (white) in study area A.

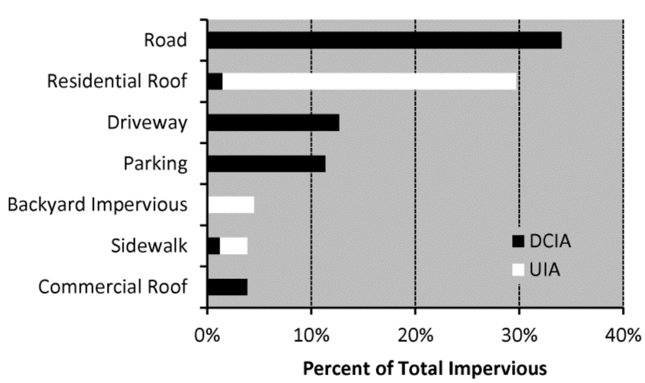

Fig. 8. Overview of types of imperviousness and respective percentages of overall impervious surface in study area A.

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Four loss scenarios were modeled in HEC-HMS using four different treatments for impervious surfaces. Curve numbers assigned to pervious portions of each model subbasin were the same for all four scenarios and ranged from 71-78 for study Area A (see CN_P, Table S3), and from 73-88 for study Area B (see CN_P, Table S4). Pervious curve numbers were calculated depending on land use and hydrologic condition of pervious surfaces for each subbasin (Cronshey et al., 1986). The scenarios differ in their treatment of impervious surfaces:

- In Scenario 1a, all impervious surfaces were included in the composite curve number for each subbasin.

$$\mathrm{CN}_{1a} = \mathrm{CN}_{\mathrm{P}} + \left(\frac{\mathrm{TIA}}{100}\right)(98 - \mathrm{CN}_{\mathrm{P}}) \tag{2}$$

  where CN_{1a} is the composite curve number for Scenario 1a, CN_P is the pervious curve number, and *TIA* is the percent total impervious area. Eq. 2 is consistent with the approach described in Technical Release 55 for connected impervious areas (Cronshey et al., 1986, Fig. 2-3 and Appendix F-1). The composite curve number is calculated as the area-average value of CN_P and TIA, whereby TIA is assigned a CN of 98.

- Scenario 1b was similar to Scenario 1a, but distinguished between directly connected and unconnected impervious areas:

$$\mathrm{CN}_{1\mathrm{b}} = \mathrm{CN}_{\mathrm{P}} + \left(\frac{\mathrm{TIA}}{100}\right)(98 - \mathrm{CN}_{\mathrm{P}})\left(1 - 0.5\left(\frac{\mathrm{UIA}}{\mathrm{TIA}}\right)\right) \tag{3}$$

  where CN_{1b} is the composite curve number for Scenario 1b, and *UIA* is the percent unconnected impervious area. Eq. 3 was adopted from Technical Release 55 for unconnected impervious areas (Cronshey et al., 1986, Fig. 2-4 and Appendix F-1); the impervious areas curve number (CN=98) is scaled by a function of the ratio of unconnected to total impervious area.

- Scenario 2 used TIA as "percent impervious" in HEC-HMS, and assigned CN_P to the pervious portion of each subbasin.
- Scenario 3 is a hybrid of Scenarios 1a, 1b, and 2. DCIA were specified as "percent impervious" in HEC-HMS. Unconnected impervious areas were included in the calculation of the composite curve number for each subbasin:

$$CN_3 = \left(\frac{CN_P * Pervious + 98 * UIA}{Pervious + UIA}\right) \tag{4}$$

  where CN_3 is the composite curve number for Scenario 3, and *Pervious* is the percent pervious area.

Curve numbers for each model scenario – by subbasin – can be seen in Table S3 (area A) and Table S4 (area B).

Transform Method

The ModClark method was selected to transform excess rainfall from each subbasin into a runoff hydrograph. Underlying the ModClark transform is a gridded representation of the watershed; each grid cell has a specified distance from the basin outlet, which is scaled by the time of concentration assigned to the subbasin (USACE, 2000; USACE, 2013a). A linear reservoir

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model accounts for storage effects within the subbasin. Model parameters (time of concentration, storage coefficient) were estimated based on GIS data and field surveys, and then calibrated using measured storm events. 10-meter and 500-meter grid files (study areas A and B, respectively) were prepared using the HEC-GeoHMS extension for ArcGIS (USACE, 2013b) based on a digital elevation model of the study area.

Results

Study Area A

Figs. 9 and 10 show a comparison of measured hydrographs (black line) and model results for 25 observed storm events in study area A. In addition to visual comparison of hydrographs, the Nash-Sutcliffe efficiency (NSE) was used to evaluate the fit of model predictions. NSE values range from $-\infty$ to 1, with 1 indicating a perfect fit between observed and modeled hydrographs (Nash and Sutcliffe, 1970). Model simulations with NSE > 0.5 are generally accepted as satisfactory (Moriasi et al., 2007).

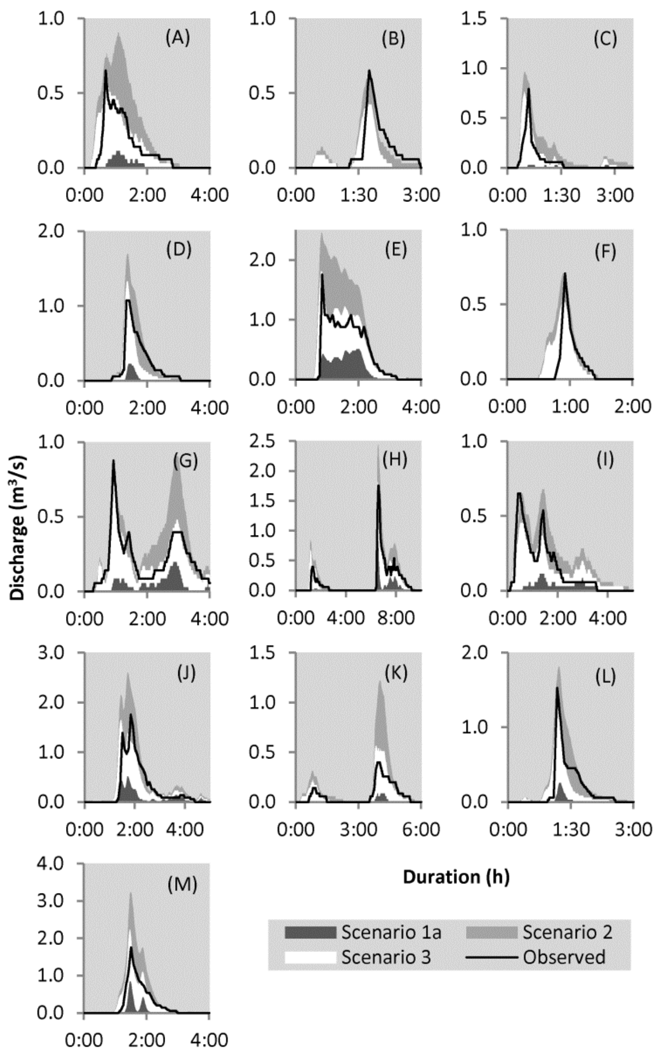

Fig. 9. Comparison of measured hydrographs (black line) and model results from Scenario 1a (dark grey), Scenario 2 (light grey), and Scenario 3 (white), for 13 observed storms in study area A.

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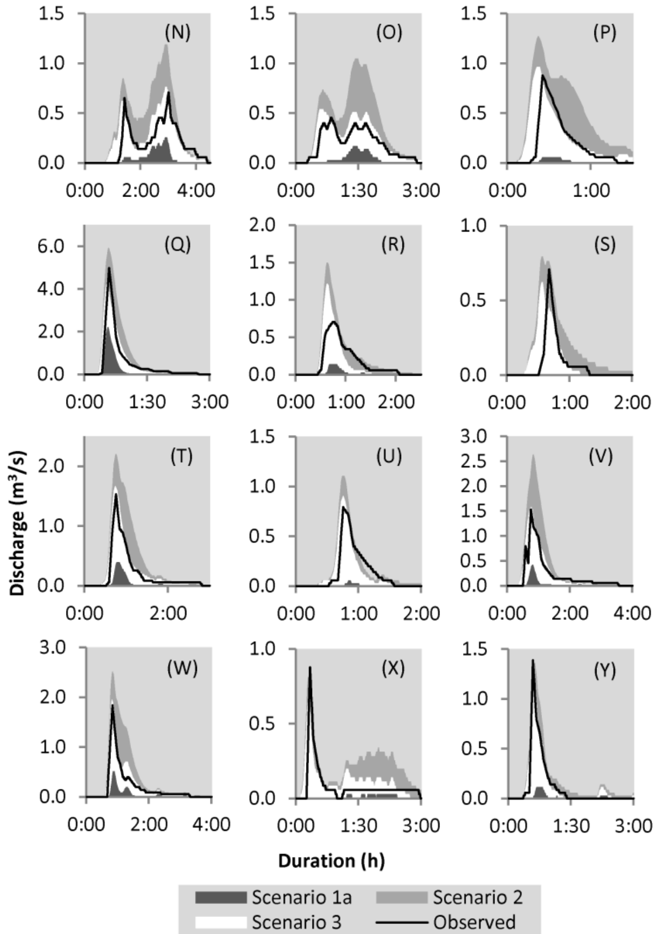

Fig. 10. Comparison of measured hydrographs (black line) and model results from Scenario 1a (darkgrey), Scenario 2 (light grey), and Scenario 3 (white), for 12 observed storms in study area A.

Scenario 1a (dark grey) underestimated measured flows in all 25 cases; with one exception (Fig. 10, "Q"), NSE values were consistently less than 0.5. Scenario 1b was excluded from Figs. 9 and 10, as results were very similar to Scenario 1a and would be difficult to distinguish on the individual hydrograph plots. Scenario 2 (light gray) overestimated observed discharge for most storms, but resulted in a good fit (NSE > 0.5) in three cases (Fig. 9, "B" and Fig. 10, "U" and "Y"). Scenario 3 (white) generated the best overall match between simulated and observed flows. In only four cases, NSE was less than 0.5 (Fig. 10, "P", "R", "S" and "X"). 21 out of 25 simulations yielded a satisfactory or good match.

Comparisons of simulated and measured peak discharges and average runoff depths from 25 storm events for all four model scenarios can be seen in Figs. 11 and 12, respectively. Scenarios 1a (cross) and 1b (dot) consistently underestimated peak discharges and runoff volumes. The opposite was true for Scenario 2 (diamond): simulated peak flows and volumes were generally higher than measured values. Model results from Scenario 3 (circle) fell closest to the line of agreement for both volumes and peak flows. Peak discharge errors for Scenarios 1a, 1b, and 2 are likely related to the large runoff volume error associated with each scenario. Owing to their small magnitude and short duration, none of the storms observed in study area A caused substantial runoff from pervious areas. For study area A with 28% DCIA and 16% UIA, losses from UIA (i.e. the difference between Scenario 2 and the line of agreement, Fig. 12) appeared to approach a constant value of approximately 4 mm.

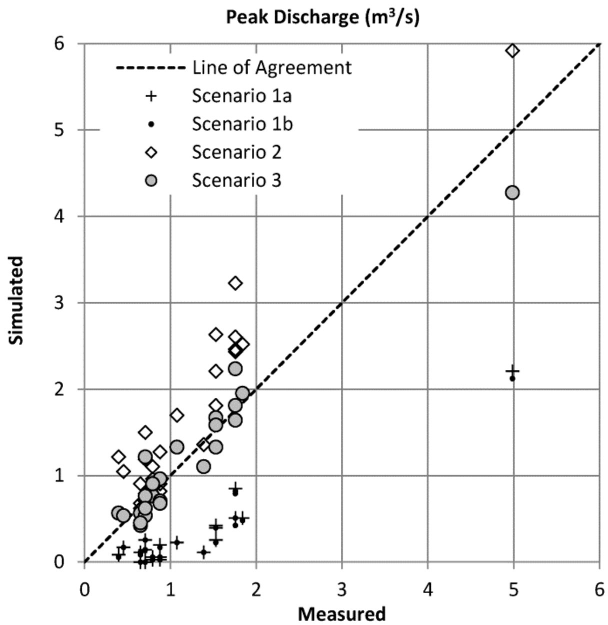

Fig. 11. Comparison of simulated and measured peak discharges for 25 storm events and four model scenarios in study area A.

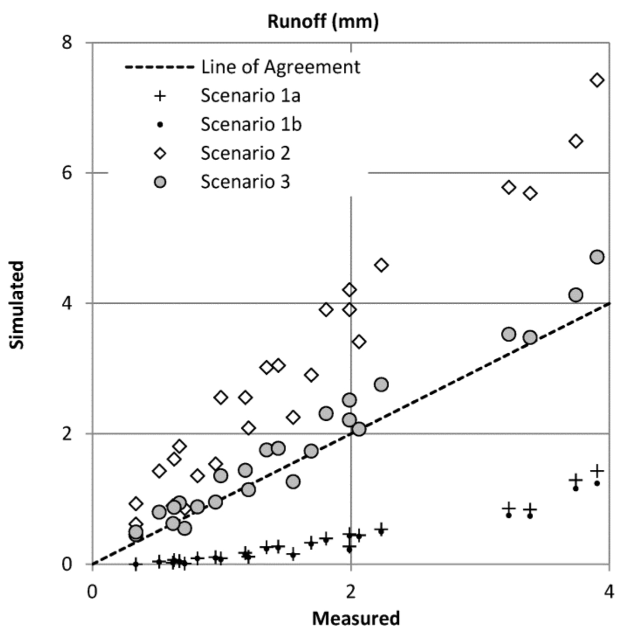

Fig. 12. Comparison of simulated and measured runoff (mm) for 25 storm events and four model scenarios in study area A.

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Study Area B

Fig. 13 shows a comparison of measured hydrographs (black line) and simulation results for 10 observed storms in study area B. Scenario 1b was excluded from Fig. 13, as results were almost identical to Scenario 1a and would be difficult to distinguish. Larger hydrograph plots for each storm are included under supplemental data alongside maps of storm extents (Figs. S2 - S11). A comparison of simulated and measured peak discharges and runoff depths are displayed in Figs. 14 and 15 (please note log scale on both axes). Simulation results from two small storms (Fig. 13, "a" and "j") show a similar pattern as in study area A: Scenarios 1a and 1b underestimated peak discharges and runoff volumes, while Scenario 2 produced results that were too high. The difference appeared to decrease with an increase in runoff magnitude. For the largest runoff events, all three scenarios generated results close to the line of agreement (Figs. 14, 15).

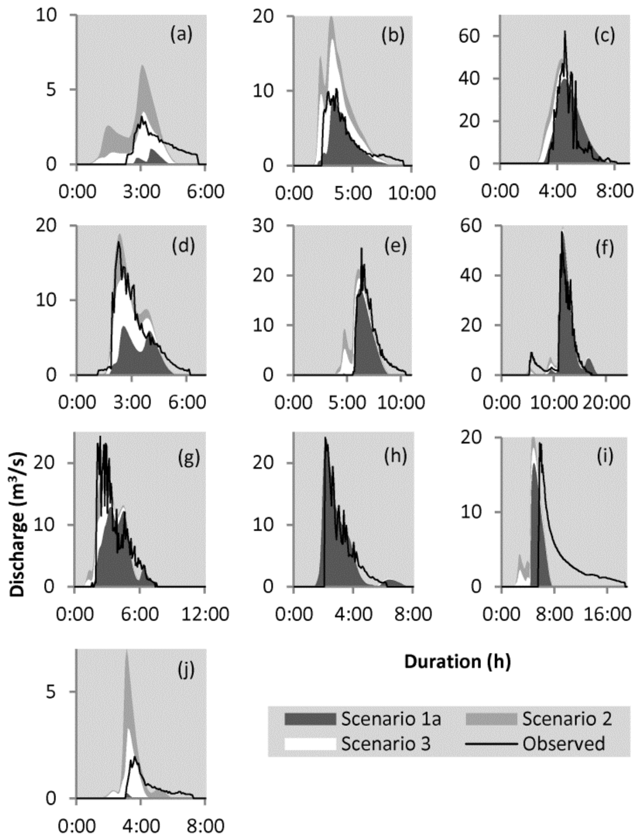

Fig. 13. Comparison of measured hydrographs (black line) and model results from Scenario 1a (dark grey), Scenario 2 (light grey), and Scenario 3 (white), for 10 observed storms in study area B.

Data from area B shows more variability than area A. For storm "b", for instance, Scenario 1a produced the best fit, while storm "i" resulted in a poor fit of simulated hydrographs for all scenarios. Some of the variability can be attributed to the complexity of the watershed, and the increased uncertainty associated with rainfall input data. NSE values exceeded 0.5 for 6 out of 10 storms for Scenarios 1a, 1b and 2. Scenario 3 showed a good match between simulated and observed hydrographs (NSE > 0.5) in 7 cases.

Overall, comparison of observed and simulated flows for area B indicate that Scenario 3 provided a better fit for small storms, but no substantial difference between scenarios was evident for moderately large storm events affecting portions of the catchment for short durations.

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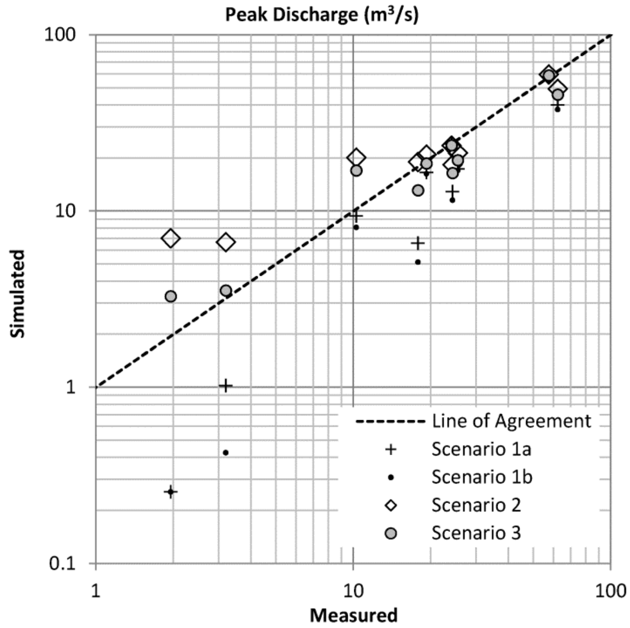

Fig. 14. Comparison of simulated and measured peak discharges for 10 storm events and four model scenarios in study area B.

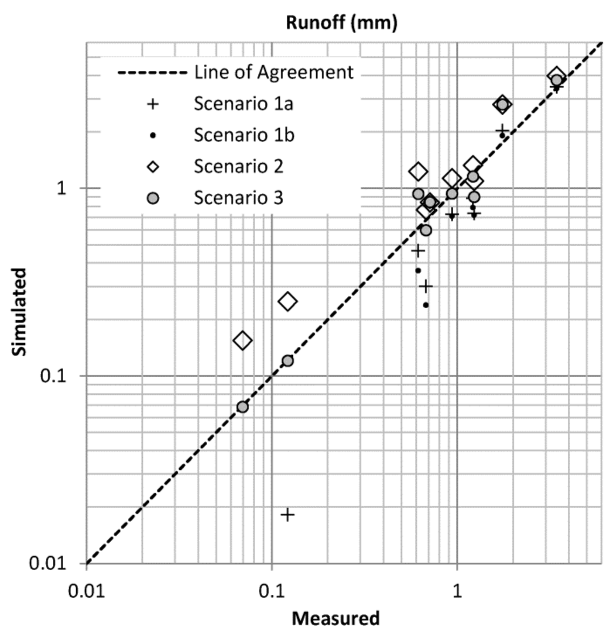

Fig. 15. Comparison of simulated and measured runoff (mm) for 10 storm events and four model scenarios in study area B.

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Discussion

Model Performance and Data Uncertainties

Discrepancies between simulated and observed hydrographs are caused – at least in part – by uncertainties in the input data. Hydrologic simulations of single events with small model time steps are more sensitive to rainfall intensities than long-term simulations (Moriasi et al., 2007). Given the fast response times of some subbasins in study area A, the corresponding model used a time step of one minute, and was therefore highly reactive to rainfall intensities. Even though high quality rain gauge data were used to develop precipitation grids, each rain gauge only provides data for one point in space. Values for areas between gauges must be obtained by interpolation, which can result in errors, particularly for storms that display large variability between gauge locations. The rain gauge network in study area B was less dense and not distributed uniformly across the watershed. Radar data had to be used to augment gauge measurements for storms that impacted the upper portion of the catchment. Rainfall estimates derived from radar, though providing a good spatial representation of the storm, are often associated with considerable systematic error (Jayakrishnan et. al, 2004; Neary et. al, 2004; Young et.al, 2000). Bias correction was applied based on the best available data sources. However, the combination of a larger watershed area, lower rain gauge density, and inherent uncertainties associated with radar data lead to an increase in the potential error of rainfall input.

Other sources of model uncertainty include variable watershed conditions such as antecedent soil moisture, temperature, and vegetative cover (USDA, 1985; Ponce and Hawkins, 1996; USDA, 2004). In this study, identical loss parameters were used for all model runs, and no adjustments were made based on antecedent moisture conditions, with one exception: storms "g" and "h" in study area B occurred less than 24 hours apart and affected largely the same location in the watershed (see Figs. S8 and S9). They were therefore modeled as one simulation run, with good results for all scenarios (Fig. 13, g and h). All remaining storms were modeled as individual events; this approach yielded satisfactory results, including for larger storms in study area B. Other studies have demonstrated that antecedent conditions can have a substantial impact on the runoff response from semiarid watersheds (Fitzjohn et al., 1998; Castillo et al., 2003; Tramblay, 2010). Any guidance developed for flood control or infrastructure design should therefore carefully consider and address antecedent conditions.

Finally, streamflow data can introduce uncertainty; very small flows that barely covered the channel bottom could not be measured accurately. Peak discharges might have been underestimated if they occurred between the 5-minute photo or transducer measurement intervals. Measured flow data from some storms in study area B (Figs. S2 – S11, storms c, d, e, f, g, and h) displayed large fluctuations in discharge near the peak of the hydrograph. This phenomenon could be caused by translatory waves (Hjalmarson and Phillips, 1997) and may reduce the accuracy of peak flow and volume measurements. In some cases, debris lines were used to adjust peak flow estimates, but debris lines were not always apparent. Finally, depth readings were converted to discharge using theoretical rating curves. The concrete channels upstream of both gauging locations are of uniform cross-section, slope and roughness, and have no confluences or obstructions. Hydraulic models show that flow in both channels is supercritical for the entire range of measured discharges. The channels are therefore well suited for development of rating curves. Theoretical ratings, however, could not be verified with direct flow measurements in the field.

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Effects of Storm Magnitude and Spatial Extent

Only storms of relatively small magnitude and duration were observed in study area A. Comparison of model simulations and observed data indicate that, during small storms, most or all of the runoff originated from DCIA; pervious and unconnected impervious areas contributed little or no flow. Data from study area A allow no conclusions relating to the infiltration characteristics of pervious areas. UIA have a measurable impact on runoff when runoff volumes are small, and should be evaluated differently than DCIA.

In study area B, only few small storms that generated runoff were observed. Small magnitude storms are often small in extent, and resulting runoff may have been below the minimum threshold (0.5 m^3/s) of the flow gauging station in study area B. Small flows have also been shown to infiltrate into the permeable bed of the main channel before they reach the outlet of the watershed (Schoener, 2016). Storms in area B varied substantially in spatial extent of rainfall distribution (Figs. S2-S11). Most storms impacted at least some portion of the urbanized area, except for storm "i" (Fig. S10), which occurred largely in the upper watershed. For the largest runoff events, data from area B showed no substantial or consistent difference between scenarios. Increased complexity of the drainage system and larger uncertainty of input parameters (specifically rainfall) lead to higher model variability. Effects of unconnected impervious areas occur early during large storms, but have a small impact on peak discharge and runoff volume; small differences due to UIA are likely lost in the overall model variability.

Data from study area B suggest that Scenario 3 produced reasonable results for the largest observed storm events, though not substantially better than Scenarios 1a, 1b, and 2. Area A results clearly show that Scenario 3 provides increased accuracy for small storms. Overall, the Scenario 3 approach of modeling imperviousness yielded the best results across a wide spectrum of storm magnitudes.

Applicability of the CN Loss Method for the Southwestern U.S.

The CN method is widely used in the southwestern U.S. (NMDOT, 1995; NDOT, 2006). The CCRFCD (1999) curve number procedure incorporates modifications to design rainfall values specific to the arid conditions of Clark County, Nevada. Some published studies question the applicability of the CN method in the Southwest. Guidance on curve number selection for western desert urban areas and arid and semiarid rangelands have been included in Technical Release 55 (Cronshey et al., 1986), but it is unclear how this guidance was developed. Hjelmfelt (1991) reported poor performance of the CN method for one watershed in the semiarid Southwest. Sabol et al. (1982) conducted runoff tests in the Albuquerque, NM area using a rainfall simulator and concluded that the curve number method is not suitable for design storms with a total depth of less than 76 mm. Heggen (1987) measured infiltration at sites in the Albuquerque area using a split ring infiltrometer and reported no strong relationship between hydrologic soil group (an important parameter in determining curve numbers), and test results. Ward and Bolton (2010) summarized data from field experiments in the Southwest and reported a weak correlation between curve numbers and measured loss rates.

Anderson et al. (2005) compared model results for a small urban basin in the city of Albuquerque with measured data and concluded that the initial and constant loss method produced superior results compared to the CN method. Their data suggest that the CN method tended to under-predict peak discharge for most storms, and runoff volume for many small storm events. In their curve number models, Anderson et al. (2005) used a composite CN that included impervious surfaces, similar to Scenarios 1a and 1b in this study. Models using the initial and

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constant loss method relied on a split hydrograph approach similar to Scenarios 2 and 3. Findings therefore appear to match results from this study, in that composite curve number simulations under-predict runoff from small storms.

Published studies indicate that the CN procedure may not be the best loss methodology for arid and semi-arid environments, particularly for small storm events where total runoff volume is a small fraction of cumulative rainfall volume. Data from study area A allow no conclusions relating to the performance of the CN method for pervious areas, since almost all runoff originated from DCIA. In study area B, scenarios that included impervious cover in composite curve numbers (1a and 1b) appeared to under-predict runoff volume for small storms, but performed reasonably well during larger storm events. It is possible that the CN loss methodology under-estimates runoff during the early part of a large storm, but represents runoff more accurately later in the event period.

The Role of DCIA

This study demonstrates the importance of accurately quantifying and modeling DCIA for small storms. Accurate estimation of DCIA for complex urban watersheds with a network of transportation infrastructure and a multitude of land use types and densities is challenging. The strategy of combining available GIS coverages, manual digitization/verification, and field surveys is time consuming and is likely not a viable option for most drainage analyses. This study confirms prior findings that transportation-related impervious areas have a greater influence on the rainfall-runoff relationship than other impervious surfaces (Schueler, 1994; Lee and Heaney, 2003). DCIA was found to be 28% of the watershed area in study area A; roads accounted for more than half of all DCIA (53%); residential driveways and commercial parking comprised 20% and 18% of DCIA, respectively. In all, 91% of DCIA in area A was transportation related. This suggests that, if time and resources are limited, special emphasis should be placed on accurately quantifying transportation related imperviousness.

It is worth mentioning that both study areas slope toward the watershed outlet, and do not include large flat areas or depressions. In urban areas with little or no slope, not all impervious coverage associated with transportation could automatically be considered directly connected. Careful assessment of local, small scale drainage patterns is necessary to avoid errors associated with over- or underestimating DCIA.

Importance of UIA

Unconnected impervious areas are separated from the drainage system by a pervious buffer. In study areas A and B, pervious buffers typically consist of desert landscaping with gravel mulch and xeric plants, or native soil; irrigated lawns can be found in some commercial and residential developments, but are generally rare. During the early phase of a storm, or during short duration storms with low cumulative rainfall amounts, all runoff from UIA is absorbed by the pervious buffer. This is evident in model simulations from study area A, where essentially all runoff from 25 observed storm events could be attributed to DCIA, while unconnected impervious and pervious areas contributed no flow. During larger storms, when direct rainfall and additional water from UIA exceed the infiltration rate of the pervious buffer, the excess becomes runoff. Loss characteristics of pervious buffers depend on many factors such as soil type and condition, plant cover, surface slope, and the dimensions of pervious area in relation to UIA that drains to it. If a large roof area drains onto a small strip of landscaping, losses from UIA will be much smaller than if the relationship is reversed. Finally, the transition between UIA and pervious

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buffer can impact losses. Water from a roof downspout draining onto a steeply sloped pervious area, for example, may lead to local erosion, causing water to quickly flow towards the drainage system in a small rill or channel without much infiltration. If, on the other hand, water from UIA is allowed to spread out over the previous surface, infiltration may be greatly increased.

Simulation runs for study area A show that disconnecting impervious surfaces in urban watersheds can substantially reduce peak flows and runoff volumes from smaller storm events. 95% of residential rooftops were estimated as unconnected. If roof runoff were captured in gutters and the gutter draining the street-facing half of the roof discharged directly onto the paved driveway, up to 50% of residential roofs could become directly connected. This would increase overall DCIA from 28 to 35% in study area A. In watersheds where a larger percentage of rooftops are connected, efforts to disconnect might start there, since disconnection can be as simple as changing the direction or point of discharge of a downspout. Overall, transportation-related imperviousness constitutes the largest share of DCIA; the focus to disconnect should therefore be on roads, parking lots and driveways. Impervious disconnection particularly benefits water quality, since the greatest impact is on smaller, frequent storm events.

Lastly, impervious disconnection makes sense on another level. Water, even urban runoff, is a limited and valuable resource. Rather than seeing it as a nuisance to be removed as quickly and efficiently as possible, it should be put to good use, sustaining plants, landscaping, and recharging local aquifers where possible. Urbanization increases the frequency and magnitude of runoff events. This can cause substantial problems, but it can also increase the supply of available water, a resource that should be planned for and utilized.

Conclusions

This case study examined the importance of impervious cover on runoff predictions from urban catchments in central New Mexico. The study demonstrates the importance of accurately quantifying directly connected impervious areas (DCIA) and unconnected impervious areas (UIA). Comparison of measured runoff and results from model scenarios with varying treatment of imperviousness for a 1.5 km^2 urban basin (study area A) showed that during storm events of short duration and small magnitude, DCIA contributed much or all of the runoff. In model simulations where all impervious areas were included in a composite curve number (Scenarios 1a and 1b), runoff was substantially underestimated. Conversely, when all impervious coverage was treated as absolutely impervious (Scenario 2), simulated runoff exceeded observed data. The best model results were achieved when DCIA was modeled as absolutely impervious (no losses, all rainfall converted to direct runoff), and UIA was included in a composite curve number (Scenario 3). Data from study area A do not allow conclusions related to infiltration characteristics of pervious areas, but support the hypothesis that UIA should be evaluated differently than DCIA. In study area A with 28% DCIA and 16% UIA, losses from UIA appeared to approach a constant value of approximately 4mm.

All scenarios were applied to a 142 km^2 watershed (study area B) and tested using observed rainfall-runoff data from storms with varying magnitudes and durations. For small storms, similar model results were observed: Scenarios 1a/b and 2 under- and over predicted peak discharge and runoff volume, respectively. The difference appeared to decrease with an increase in runoff magnitude. For the largest runoff events, all three model scenarios produced results close to observed data. Overall, comparison of observed and simulated flows for area B indicated that Scenarios 2 and 3 provided a slightly better fit compared to Scenarios 1 a and b, but no substantial difference between Scenarios 2 and 3 was evident.

Treating DCIA as impervious, and including UIA in a composite loss computation, appears to be a modeling approach that yields satisfactory results for storms of different magnitudes and durations. One of the ensuing benefits is that hydrologic simulations built with flood control in mind may be used to simulate runoff from smaller magnitude storms, e.g. to assess the impacts of urbanization on water quality.

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Supplemental Data

Tables S1-S4 and Figs. S1-S11are available online in the ASCE library (ascelibrary.org).

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