Sensitivity of one-dimensional hydrologic model simulations: A model study of Lemes Canyon, New Mexico

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Sensitivity of One-Dimensional Hydrologic Model Simulations: A Model Study of Lemes Canyon, New Mexico

By

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B.S. Environmental Science

Professional Project
Submitted in Partial Fulfillment of the Requirements for the Degree of
Master of Water Resources

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May 2016
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Sensitivity of One-Dimensional Hydrologic Model Simulations: 
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Abstract
Stream channel restoration can increase flow storage and energy dissipation of passing flood waves (Sholtes and Doyle, 2011). Reestablishment of floodplain hydraulic function and increasing floodplain connectivity are increasingly goals of restoration programs, yet the magnitude of possible change to a range of variables remains poorly quantified (Bernhardt and Palmer, 2011; Sholtes and Doyle, 2011).

This study utilizes HEC-RAS to route floods under steady state, subcritical conditions in field-based impaired reach scale models. The study integrates collected channel data from Lemes Canyon, an ungaged ephemeral channel located near Monticello, NM as well as USGS topographic data (7.5 minute; 1:24,000) to construct a model at the reach scale. Peak discharge values were estimated using the USGS Generalized Least Squares Regression for Ungaged Streams. This study performed sensitivity analyses of one-dimensional hydrologic model simulations to quantify the magnitude of change with respect to two response variables, average total velocity and hydraulic depth, respectively.

In this study synthetic ineffective flow areas were used as a proxy for engineered log jams to test the hypothesis of equal population means against the alternative that not all population means are equal for the two response variables among ten geometric plans. A One Way Analysis of Variance (ANOVA) of means among populations was performed to test the hypothesis for both response variables. At the .05 level, no statistically significant results were found. The results
from this study indicate there are no statistically significant differences in mean values with respect to the two response variables among all ten populations considered.

These results suggest there is no statistical evidence that ineffective flow areas as a proxy for log jams are effective at decreasing the average velocity or increasing the hydraulic depth at the reach scale. The statistical results identify the relative importance of hydrologic design elements used in channel reconfiguration projects among ephemeral and intermittent channels in arid and semi-arid climates.

**Keywords:** Steam channel restoration, one-dimensional, average velocity, hydraulic depth, log jams, ineffective flow areas, ephemeral
Introduction

River Restoration is an increasingly common approach utilized to reverse past degradation of freshwater ecosystems and to mitigate the damage to watersheds from human activities (Bernhardt, E. & Palmer, M., 2011). As discussed in Walsh et al. 2005 and Bernhardt & Palmer 2007, human activities leading to non-point-source pollution and channel degradation are among the most common motivations for undertaking stream restoration, often involving significant channel reconfiguration efforts. Bernhardt and Palmer (2011) suggests that these channel-based or “hydro-morphological” restoration projects occur worldwide (Jähnig et al. 2009), and point to recent research efforts to evaluate their ecological effectiveness (Tullos et al., 2009; Baldigo et al., 2010; Miller and Kochel, 2010).

Stream channels are restored to meet a variety of goals. These goals may include maintaining water quality, providing habitat for aquatic species, and storing and attenuating flood flows [Federal Interagency Stream Restoration Working Group (FISWRG) 1998]. Literature suggests there are many benefits associated with channel restoration. As cited in Sholtes and Doyle (2011), one of the potential benefits of channel restoration is the reversal of the effects of channelization and incision by restoring the ability of the storm channel and floodplain to slow down and retain flood waters (Acreman et al., 2003; Campbell et al., 1972; Liu et al., 2004). Retaining floodwater or encouraging inundation through restoration or intervention may enhance inundation of floodplains and floodplain diversity. Floodplain diversity and production has been attributed to the dynamic and variable connectivity with river flows, where periods of inundation and high flows are interconnected to high floodplain productivity Junk et al., 1989). This study presents an analysis of the sensitivity of log jams for
channel restoration through use of one-dimensional steady state hydrologic model simulations. The study explores engineered log jams as a channel restoration technique at the reach scale.

The current regulatory framework for performing restoration activities in ungauged ephemeral and intermittent streams can be described as layered and occurring at different scales. The regulatory framework encompasses both federal and state agencies. The institutional arrangements comprising the regulatory framework include both federal common law and state statute. The regulatory agencies responsible for compliance of water quality and geomorphological conditions of ephemeral channels are the New Mexico Environment Department (NMED) and the United States Army Corps of Engineers (Corps), respectively.

The Water Quality Act establishes the Water Quality Control Commission (WQCC). The Water Quality act provides authority for water quality management to the NMED. The WQCC is the state water pollution control agency for purposes of the Federal Clean Water Act (1972), administered by the United States Environmental Protection Agency (USEPA). Through state statute the NMED is responsible for implementing the Federal Clean Water Act in New Mexico and ensuring surface waters meet state water quality standards.

The regulatory framework requires compliance with Nationwide Permits (NWP) under section 404 of the Clean Water Act. Specifically, conducting activities in ephemeral or intermittent streams requires compliance with NWP No. 27 and compliance with General Conditions No. 25 (Water Quality) and No.31 (Pre-Construction Notification) where the certifying agencies are the NMED Surface Water Quality Bureau and the Corps, respectively.

This model study aims to increase the understanding of the how two response variables, average total velocity (cross section) and hydraulic depth (cross section), respectively are
affected by the integration of synthetic flow areas into comparable, geometric plans under the same flow conditions (2-year flood). Both response variables operate at a localized scale (cross section) and may help explain the likelihood of success when placed in the context of restoring impaired channels for the purpose of improving hydrological function including floodplain connectivity.

**Study Site**

The study site, Lemes Canyon, is an unaged ephemeral reach within the Garcia Falls-Alamosa Creek Watershed, 12-Digit Hydrologic Unit Code (130202110703) near Monticello, NM (See Map 1). This subbasin of the Elephant Butte Watershed (8-digit HUC 13020211) lies within the Mexican Highlands section of the Great Plains Physiographic Section. This region is defined by north-south trending, isolated mountain ranges separated by aggraded desert plains (Sierra Soil and Water Conservation District, 2008). The study site is an ephemeral channel that is dry for most of the year. The channel is unaged. Monsoon rains and other rain events drive unaged channel flows. Forty-nine farms in the immediate area irrigate approximately 800-acres of land from Alamosa Creek, additional wells, and occasionally from its floodwaters (Alamosa Land Institute, 2011). The Subbasin of interest straddles Sierra County and Socorro County. Lemes Canyon is located in Sierra County that flows into Alamosa Creek. Research and analysis will be performed at the reach-scale on Lemes Canyon (approximate reach size 2.5 km).

Historic dry periods over the past two-hundred years along with grazing pressure decreased vegetation both in the riparian area and uplands of the watershed. Grassland was further stressed by competition from woody shrubs and trees historically kept in check by natural fires that are now suppressed (Alamosa Land Institute, 2011). These natural and anthropogenic disturbances have created landscape conditions which are associated with high energy flows of
flood waters during storm events. Ultimately, a watershed scale restoration plan would integrate upland and lowland restoration activities to address degradation that has led to impaired channels and streams within the study site.

Figure 1 This map illustrates the project study site within the 12-Digit HUC.
Research Question

This study is an analysis of the sensitivity of steady state, one-dimensional, hydrologic model simulations, using field and topographic data under different estimated peak discharge values to ineffective flow areas. In the HEC-RAS environment ineffective flow represents areas where flow velocities are very low or approaching zero (i.e., areas having a combination of flow velocities less than 0.5 feet per second and depths less than three feet) (US Army Corps of Engineers, Hydrologic Engineering Center, 2010).

Steady flow describes conditions in which depth and velocity at a given channel location do not change with time. In this study synthetic ineffective flow areas are used as a proxy for engineered log jams. Engineered log jam technology is founded on the premise that river management can be improved by understanding, mimicking and that have the potential to augment natural hydrologic processes (Abbe et al., 2003). Engineered Log Jam is a term used commonly in the restoration community and is synonymous with the term constructed log jam(s).

Ten geometric plans were created in the HEC-RAS environment. Each plan was modeled under the estimated 2-year peak discharge. The plans ranged from a baseline condition (no ineffective flow areas) to a hypothetical maximum condition that integrated twelve synthetic flow areas. The response variables analyzed were average total velocity (cross section) and hydraulic depth (cross section).

The research question this study attempts to answer is whether there are any statistically meaningful changes in the mean values for average velocity (cross section, m/s) and hydraulic depth (cross section, m) among populations. Populations (groups) are defined by the degree of engineered log jams integrated. The one-dimensional model simulation output for steady state
flows are compared to a baseline condition as well as nine other hypothetical channel restoration scenarios. All nine scenarios (not including the baseline; zero log jams) were compared for statistically significant changes to the selected response variables.

A sensitivity analysis was performed utilizing global techniques that concentrate on estimating the local impact of a hydrologic parameter on the model output. The study documents the sensitivity of average total velocity (cross section) and hydraulic depth (cross section by routing estimated peak discharge (2-year flood) through the impaired reach. The sensitivity analysis was performed by making global adjustments to Manning’s $n$ a channel roughness parameter, in the HEC-RAS Environment. The sensitivity analysis compared the current impaired field-based reach under baseline conditions with conditions that are equal to $\pm$ 30% of the baseline roughness parameter value.

**Hypothesis**

This study aims to learn whether the magnitude of change in response variables, specifically average total velocity (cross section) and hydraulic depth (cross section) is dependent on the type of imposed channel flow conditions defined by the total number of ineffective flow areas. The populations (groups) are comprised of fifty-one cross sections (eight field based cross sections with forty-three interpolated cross sections, spaced at 10m intervals). There were a total of 510 observations. Population means for both response variables were compared for variance in means for flows associated with the estimated 2-year flood ($13.04 \text{ m/s}$).

In notation the hypothesis is stated:

$$H_0 : \mu_1 = \mu_2 = \mu_3 = \mu_4 \ldots \mu_{10} \text{ against } H_1 : \mu_1 \neq \mu_1 \neq \mu_1 \neq \mu_1 \ldots \mu_1 \text{ or not } H_0$$

Where $H_0$ is the null hypothesis and where $H_1$ is the alternative hypothesis

Where $\mu$ = the average total velocity (cross section) or the hydraulic depth (cross section)

Let $\alpha = .05$ (level of significance)
In words, the null hypothesis states that there will be no change in the response variables *average total velocity (cross section) and hydraulic depth (cross section)*, respectively when compared among all ten groups, for the 2-year estimated peak discharge value at the $\alpha$ level. The alternative hypothesis states that there will be change in at least one of the considered groups at $\alpha$ level.

**Methods**

**Response Variables**

The primary response variables that were used in this study to quantify sensitivity of this one-dimensional model were changes in, 1) average total velocity, defined as the flow divided by the area of the cross section, and 2) hydraulic depth, which is defined as the areas of cross sectional flow divide by the wetted perimeter (HEC-RAS defines this as the area/top width of flow). Both response variables are standard outputs of the HEC-RAS modeling environment and are calculated at each cross section within the model. HEC-RAS compute both response variables user defined areas or slices (channel, left over bank, right over bank) of each cross section.

The study compares the mean values for each response variable among ten geometric plans (groups). Plans integrate synthetic ineffective flow areas as a proxy for engineered log jams. The analyses are performed under steady state conditions, are subcritical in nature, and are reported for the 2-year flood. To summarize the range of analysis, the number of constructed log jams varies by plan (i.e. group), the channel position is curved slightly, and all analyses are performed under the 2-year flood condition.

Where:

*Average Velocity Total*
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\[ V = \frac{Q}{A} \]  \hspace{1cm} (Equation 1)

Where:

\( V \) = Velocity
\( Q \) = Volumetric discharge
\( A \) = Cross sectional area of flow

By using the Continuity Equation the mean velocity at a given cross section where user defines “slices” or regions that define the compound channel. The Continuity Equation is expressed as \( \rho A_1 V_1 = \rho A_2 V_2 \) for steady one-dimensional flow, non-sediment laden flows.

**Hydraulic Depth**

\[ h_m = \frac{A}{T} \]  \hspace{1cm} (Equation 2)

For use in Froude number and energy relationships in open channel flow hydraulics, mean depth, \( h_m \), is defined as the depth which, when multiplied by the top water surface width, \( T \), is equal to the irregular section area, \( A \), is commonly used for critical flow relationships (Dodge 2001). The equation for hydraulic mean depth, \( h_m \), is:

**Model Infrastructure**

This study utilized three software environments ArcGIS 10.1, HEC-RAS\(^1\) and HEC-GeoRAS. The software environment GeoRAS was used initially to digitize feature class datasets. These feature classes (i.e. stream channel, stream centerline, banks, and flow paths) were imported into HEC-RAS for modeling and analysis. Four types of data were necessary to perform this study: 1) 10m High Resolution Digital Elevation Model (DEM), 2) Digital Orthophotography, 3) Field Data (geometric), and 4) USGS Topographic data (7.5 minute).

Field data were used as input in the HEC-RAS environment once cross sections were defined and georeferenced. Digital Orthophotography were used along with DEM to locate the channel, banks and floodplains. One of the key advantages to using orthophotos is that relief displacement

\(^1\) This study will use [HEC-RAS version 4.1; U.S. Army Corps of Engineers (USACE) 2008], HEC-RAS is widely used within the channel restoration design community; and HEC-RAS modeling, or a comparable one-dimensional model, is often required as part of channel restoration designs (Sholtes and Doyle 2011).
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has been removed so that ground features are preserved in their true locations. Digital Orthophotography such as NAIP imagery is sometimes used in scientific analysis especially when the direct measurement of angles, distances, areas, and positions of objects or landscape features may be necessary.

Figure 2 (Left; a) A HEC-GeoRAS representation using digital orthophotography from the National Agriculture Imagery Program (NAIP) is used to visualize extent of the lower reach (approximately 2.6 km) of Lemes Canyon not including the upper drainage area (see figure 6) Figure 3 (Right; b) A zoomed view highlights digitized feature classes within the study site including the locations of field based cross sections (green).

Visual inconsistencies in the DEM and the Orthophotography were noted. Reasons for these discrepancies could be related to processing of raw DEM data, changes in geomorphology that occurred between the times the remote sensing data were collected, to name a few. The datum for the spatial datasets were D North American 1983. The spatial reference or “projections” used were NAD 83 Universal Transverse Mercator (UTM) Zone 13. The DEM dataset provided an array of elevations relative to ground positions. In HEC-GeoRAS it is
necessary to provide a DEM or grid or a Triangulated Irregular Network (TIN) as reference of which geometric based feature classes are created.

**Steady State Flow Analysis**

This project employed HEC RAS to model one-dimensional flow under steady state conditions. Steady state flow conditions assume that depth and velocity at any location in the stream do not change with time. Under these conditions, the study aims to shed light on how engineered log jams, modeled as ineffective flow areas, affect the response variables.

HEC-RAS calculates one-dimensional water surface profile for steady gradually varied flow in natural or constructed channels (Army Corps of Engineers, 2010). This study used a subcritical flow regime to define steady state conditions. Under steady state conditions, HEC-RAS computes water surface profiles from one cross section to the next, downstream to upstream, by solving the energy equation in an iterative manner known as the “step-up” method (Army Corps of Engineers, 2010, p. 2-2). The energy equation used for open channel flow is a simplification of the Bernoulli Equation.

The Energy Equation can be expressed as follows:

\[
Z_2 + Y_2 + \frac{a_2 + v_2^2}{2g} = Z_1 + Y_1 + \frac{a_1 + v_1^2}{2g} + h_e
\]  

(Equation 3)

Where:

- \(Z_1, Z_2\) = elevation of the main channel inverts
- \(Y_1, Y_2\) = depth of water at cross sections
- \(V_1, V_2\) = average velocities (total discharge/ total flow)
- \(a_1, a_2\) = velocity weighting coefficients
- \(g\) = gravitational acceleration
- \(h_e\) = energy head loss
As discussed in Army Corps of Engineers (2010), the following assumptions are implicit in the analytical expressions used in the version HEC-RAS 4.1, 1) flow is steady, 2) flow is gradually varied, 3) flow is one-dimensional (velocity components other than in the direction of flow are not accounted for), and 4) river channels have slopes of less than 5.71 degrees or 1:10.

To examine the sensitivity of one-dimensional models to imposed hypothetical flow resistance conditions that integrate synthetic ineffective flow areas as a proxy for engineered log jams are simulated. The study tests the hypothesis of equal population means against the alternative that not all population means or medians are equal. Informal and formal statistical tests were performed for both response variables among all ten groups (geometric plans), each containing fifty-one cross sections from both field and interpolated data.

The study has two primary analytical components, 1) steady state analysis, 2) and a sensitivity analysis of Manning’s n. Additional work was performed to characterize other prevailing hydrologic conditions. Specifically, the results of a falling head permeability test are reported in the results section but no statistical analysis was performed. Informal and formal tests of normality were performed on the selected model steady state simulation output. Boxplots, histograms, violin plots, bootstrap sampling of the mean, and a QQ Test were all used to initially assess the spread, range and distribution of the data. Two formal tests of normality were performed, the Shapiro-Wilk Test and the Anderson-Darling Normality Test, respectively.

A One Way Analysis of Variance (ANOVA) was performed among equal population means against the alternative that not all population means are equal. The classical ANOVA test assumes that the populations have normal frequency curves and the populations have equal variances. The ANOVA uses the F-Statistic to calculate variance in means among populations. Statistical analysis of the select data was performed in the environment “R”. The sensitivity and
flood inundation component of this study did not require statistical analysis, its purpose is contextual.

Cross Section Data

The one-dimensional model was constructed using channel survey data as well as interpolated cross section data. Cross section data was gathered in the field, during times when the channel was dry, November 2014 and June 2015, respectively. Eight cross sections were completed in the field. Downstream cross sections numbers 1-4 (1, being the furthest downstream location) were spaced approximately 100m apart. Upstream cross sections were spaced slightly closer together to capture floodplain characteristics, located channel left.

Figure 4 A screenshot view from the HEC-RAS user interface illustrating locations of eight field-based cross sections, digitized channel centerline (blue). Direction of flow is noted by the black arrow adjacent to the stream centerline.

Cross section data were measured and collected using the rod and level method. Data from these eight cross sections were integrated into the HEC-GeoRAS and then HEC-RAS to develop the steady state modeling spatial framework (Figures 1 and 2). Actual field data was preferred because it provides a finer resolution than currently available raster data.
Figure 5 Exported cross section from the HEC-RAS cross section editor. For illustrative purposes the cross section, looking downstream, shows the water surface elevation for six flow profiles (annual exceedance probability; .02, .05, .1, .25, .5, 1) where the dark blue represents the water surface elevation at the .02 or 2-year flood volumetric flow rate.

The methodology employed to collect field data is explained in the USDA Forest Service General Technical Report RM-245 (Harrelson et al., 1994). Channel survey with a recorded datum and coordinate system provide an opportunity for replicate surveys in the future. An established record will enable detection of future geomorphic change that might occur as a result of flood scour, bed-material aggradation, or lateral channel migration (Emmett and Hadley, 1968).

In this study, interpolated cross sections were defined at 10m interval spacing (Figure 3). A total of forty-three interpolated cross sections were included from downstream to upstream. In HEC-RAS interpolated cross sections are necessary when the velocity head is too large to
accurately determine the change in energy gradient (Army Corps of Engineers, River, 2010). Cross sections were also defined by including conveyance obstructions in areas where flow is known not to occur under specified flows (Figure 3). In HEC-RAS obstructions decrease flow area but add wetted perimeter when the water comes into contact with them. When a user adds an obstruction the water is not prevented from going outside the obstruction.

Figure 6 Exported cross section from the HEC-RAS cross section editor. For illustrative purposes the black area shown here represents a user-defined conveyance obstruction. Conveyance obstructions allow the user to define areas at a cross section that are permanently blocked from conveying flow. Conveyance obstructions decrease flow area and add additional wetted perimeter where the water comes in contact with the obstruction.
Figure 7 A screenshot view of the one-dimensional steady state model built in HEC-RAS. Eight field based cross sections along with forty-three interpolated cross sections at 10m are shown. Black areas extending beyond the green cross sections on channel right are representative of obstructed flow areas. Bank stations are shown here as red dots along either side of the stream channel centerline (blue).

To better represent both the channel and adjacent floodplain areas within the study site the cross sections needed to be extended longitudinally in the modeling environment. This was done by bringing USGS 7.5 minute (1:24000) topographic maps into the ArcGIS (HEC-GeoRAS) environment and extrapolating elevation values where changes in elevation occur within the floodplain. In this study the available 10m DEM was not appropriate because there were discrepancies in elevation values when compared to collected data. This limited the ability to perform spatial analysis and to integrate the simulated inundation results into the HEC-GeoRAS environment for inundation mapping.

Ineffective flow areas were added to selected geometric plans so that the sensitivity of the one-dimensional model could be tested against a baseline. The location of ineffective flow areas (i.e. synthetic engineered log jams) was determined based on private GIS datasets that contain feature classes for both existing and hypothetical locations of engineered log jams in Lemes
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Canyon. Ineffective flow areas have been placed in channel within banking stations, near the left bank (Figure 4). Ineffective flow areas are spaced 10m apart for simulations. The sizes of the ineffective flow areas range in this model. The approximate dimensions of ineffective flow areas in this model range from .5m-1.5m in height to 6m-10m in length.

Figure 8 Exported cross section from the HEC-RAS cross section editor shows the addition of a synthetic ineffective flow area (inside the red circle; green outlined triangle with diagonal striping) located within the main channel. In this study ineffective flow areas were used as a proxy for engineered log jams. This triangular shape takes on the contour of the channel bottom and is defined by an elevation.

Modeling Plans

Within the HEC-RAS environment the user has ability to formulate several different geometric plans. These user defined plans can represent different sets of geometric data and flow data. In this study sixteen geometric plans have been defined (See Table 1), and six-flow profiles have been defined (See Table 2). For this study sixteen unique user defined plans were created. A plan description as well the presence of ineffective flow areas and their relative location within the lower reach of Lemes Canyon are displayed below. Geometric plans were
created for the two purposes, 1) for quantifying the sensitivity of one-dimensional flows relative to varying numbers of ineffective flow areas, and 2) for reporting mean value outcomes with respect to response variables.

Table 1 User defined geometric plans in the HEC-RAS environment

<table>
<thead>
<tr>
<th>Plan Number</th>
<th>Plan Description</th>
<th>Ineffective Flow Areas</th>
<th>Location of Ineffective Flow Areas (distance from first downstream station meters)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plan 01</td>
<td>Baseline Conditions</td>
<td>N</td>
<td>NA</td>
</tr>
<tr>
<td>Plan 02</td>
<td>Plus 10% Manning’s n</td>
<td>N</td>
<td>NA</td>
</tr>
<tr>
<td>Plan 03</td>
<td>Plus 20% Manning’s n</td>
<td>N</td>
<td>NA</td>
</tr>
<tr>
<td>Plan 04</td>
<td>Plus 30% Manning’s n</td>
<td>N</td>
<td>NA</td>
</tr>
<tr>
<td>Plan 05</td>
<td>Minus 10% Manning’s n</td>
<td>N</td>
<td>NA</td>
</tr>
<tr>
<td>Plan 06</td>
<td>Minus 20% Manning’s n</td>
<td>N</td>
<td>NA</td>
</tr>
<tr>
<td>Plan 07</td>
<td>Minus 30% Manning’s n</td>
<td>N</td>
<td>NA</td>
</tr>
<tr>
<td>Plan 08</td>
<td>1 Log Jam Modeled</td>
<td>Y</td>
<td>378.497</td>
</tr>
<tr>
<td>Plan 09</td>
<td>2 Log Jams Modeled</td>
<td>Y</td>
<td>368.716</td>
</tr>
<tr>
<td>Plan 10</td>
<td>3 Log Jams Modeled</td>
<td>Y</td>
<td>358.935</td>
</tr>
<tr>
<td>Plan 11</td>
<td>4 Log Jams Modeled</td>
<td>Y</td>
<td>349.154</td>
</tr>
<tr>
<td>Plan 12</td>
<td>5 Log Jams Modeled</td>
<td>Y</td>
<td>339.373</td>
</tr>
<tr>
<td>Plan 13</td>
<td>6 Log Jams Modeled</td>
<td>Y</td>
<td>329.591, 319.810, 310.029</td>
</tr>
<tr>
<td>Plan 14</td>
<td>8 Log Jams Modeled</td>
<td>Y</td>
<td>319.810, 310.029, 300.248, 290.467</td>
</tr>
<tr>
<td>Plan 15</td>
<td>10 Log Jams Modeled</td>
<td>Y</td>
<td>300.248, 290.467, 280.686, 271.427</td>
</tr>
</tbody>
</table>

Table 2 Estimated peak discharge values at standard recurrence intervals

<table>
<thead>
<tr>
<th>Flow Profile</th>
<th>Volumetric Discharge (m^3)</th>
<th>Recurrence Interval (RI), in Years</th>
<th>Percent Chance of Recurrence (in any given year)</th>
<th>Annual Exceedance Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>13.04</td>
<td>2</td>
<td>50</td>
<td>0.02</td>
</tr>
<tr>
<td>2</td>
<td>25.60</td>
<td>5</td>
<td>20</td>
<td>0.05</td>
</tr>
<tr>
<td>3</td>
<td>36.57</td>
<td>10</td>
<td>10</td>
<td>0.1</td>
</tr>
<tr>
<td>4</td>
<td>53.68</td>
<td>25</td>
<td>4</td>
<td>0.25</td>
</tr>
<tr>
<td>5</td>
<td>68.98</td>
<td>50</td>
<td>2</td>
<td>0.5</td>
</tr>
<tr>
<td>6</td>
<td>86.42</td>
<td>100</td>
<td>1</td>
<td>1.00</td>
</tr>
</tbody>
</table>
Flow Profile Data: Estimating Peak Discharge Rates

In this study six flow profiles, representative of estimated volumetric flow rates at standard return intervals were calculated. Only the 2-year flood estimate was utilized in analyses. A synthetic hydrograph based on the USGS Regional flood-Frequency Equations using the Generalized Least-Squares Regression was generated. Lemes Canyon is located in USGS Flood Region 7 (New Mexico; for additional information refer to Waltemeyer, S. D., 2008). As a check, the National Streamflow Statistics (NSS) Model was also used to calculate the peak discharge. In the NSS model the study site is within the Crippen and Bue Flood Region 16. The two models utilized similar regressions to estimate peak discharges. This model integrates the estimated peak discharge values from the Generalized Least-Squares Regression.

**USGS Regional Flood Frequency Equation, Peak Discharges**

*Generalized Least-Squares Regression*

\[ Q_n = \text{regression coefficient} \times 10^2 \times A^n \]  

*(Equation 4)*

- \( n \) = Return interval
- \( A \) = Drainage area of the basin

\( Q_{pk} = Q_2, Q_5, Q_{10}, Q_{25}, Q_{50} \ldots \) peak discharge value at varying recurrence intervals

\[
\begin{align*}
Q_2 &= 1.465 \times 10^2 \times A^{454} \\
Q_5 &= 2.777 \times 10^2 \times A^{468} \\
Q_{10} &= 3.878 \times 10^2 \times A^{477} \\
Q_{25} &= 5.537 \times 10^2 \times A^{488} \\
Q_{50} &= 6.955 \times 10^2 \times A^{497} \\
Q_{100} &= 8.518 \times 10^2 \times A^{506} \\
Q_{500} &= 1.275 \times 10^3 \times A^{529}
\end{align*}
\]

Where

- \( Q \) represents peak discharge, in cubic feet per second, for indicated recurrence interval; \( A \), drainage area, in square miles.

An area for the Lemes Canyon drainage area was estimated by performing flow direction, flow accumulation and watershed delineation in ArcGIS (10.1). Using flow direction output as a
guide, a smaller catchment for Lemes Canyon was digitized as a polygon feature class (Figure 5). From this new feature class an area for the drainage was calculated (Figure 6). This estimation of drainage area was used as an input to the USGS Regional flood-frequency equations using generalized least-squares regression (Waltemeyer, 2008). The area defined by running the ArcGIS model was compared to the National Hydrologic Dataset, HUC-12 shapefile for accuracy.

Figure 9 (Left), ArcGIS (10.1) image of the modeling results from the flow direction analysis where subbasin was delineated. Colors are representative of standard flow direction analysis using raster data. Figure 10. (Right) a new feature class was created from the flow analysis. The area of beige colored subbasin was calculated as 32.37 square kilometers (12.5 square miles) and used as in input into regression models to calculate estimated peak discharges.

Table 3 Overview of basin characteristics

<table>
<thead>
<tr>
<th>Study Reach</th>
<th>Upstream Drainage Area (km²)</th>
<th>Downstream Drainage Area (km²)</th>
<th>Strahler Stream Order</th>
<th>Channel Length (m)</th>
<th>Slope (m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lemes Canyon</td>
<td>32.37</td>
<td>3.55</td>
<td>3</td>
<td>469</td>
<td>0.023</td>
</tr>
</tbody>
</table>

Manning’s Roughness Coefficient n

Manning’s n, a roughness coefficient needed to be defined within each channel and floodplain, cross sectional area of the model. Manning’s n values for channel, left bank and right bank were
estimated using a formula developed by Cowan (1956). Cowan developed a procedure for estimating the effects of these factors to determine the value of n for a channel.

\[ n = (nb+n1 +n2 +n3 +n4) m \]  

from Cowan (1956)

Where

- \( n_b \) = a base value of n for a straight, uniform, smooth channel in natural materials,
- \( n_1 \) = a correction factor for the effect of surface irregularities,
- \( n_2 \) = a value for variations in shape and size of the channel cross section,
- \( n_3 \) = a value for obstructions,
- \( n_4 \) = a value for vegetation and flow conditions, and
- \( m \) = a correction factor for meandering of the channel.

Estimation of Manning’s n values for vegetated channels in arid to semi-arid environments can present difficulties in estimating the channel’s resistance to flow (Phillips and Tadayon, 2006). For example, with respect to ephemeral and intermittent streams located in arid and semi-arid regions vegetation may change considerably over a period of time or during a flood event. Determination of Manning’s n requires acquired skill, judgment, field expertise and in many respects can be thought of as an art (Chow 1959; Barnes 1967; Limerinos 1970). For this research I have followed the procedures for the selection of Manning’s roughness coefficient for natural vegetated and non-vegetated channels as discussed in Phillips and Tadyon, 2006; and Cowan 1956).

**Slope**

When running a steady state simulation in HEC-RAS it is necessary to define the normal depth (channel bed slope) or slope as a boundary condition. Channel bed slope was calculated from collected channel survey data. Channel bed slope is defined as the change in elevation over a linear distance, or rise over run (reported units, m/m).
Sensitivity Analysis

Sensitivity analysis is recognized as an important aspect of the use of hydraulic models (Hall et al., 2009). To account for uncertainties in one-dimensional modeling, using HEC-RAS, global variance-based sensitivity analyses are sometimes used. Often in hydraulic modeling a global variance-based sensitivity analysis has been shown to be more general in its applicability and in its capacity to reflect nonlinear processes and the effects of interactions among variables. For simplicity, this research applies global changes to Manning’s n values before simulated runs.

The general framework used by Sholtes and Doyle (2011) to conduct a sensitivity analysis will be utilized for this project. A sensitivity analysis of water surface elevation and average total velocity to individual channel and floodplain properties will be conducted using the field-based impaired reach model as a baseline. Modeled differences due to ±30% changes from the baseline morphology will be reported in terms of average total velocity, hydraulic depth and water surface elevation.

Falling Head Permeability Test of Soil

Modeling flow in an ephemeral stream with a sand bottom channel requires knowledge of the channel's infiltration rate. Double-ring infiltrometers are used by soil scientists and other professionals to measure the infiltration rate (Gregory et al., 2005). One of the advantages of the falling-head test over the conventional constant-head test is its ability to determine permeability properties of the test material, in this case soil, at different levels of hydraulic gradients in a single test (Fwa, Tan and Chuai, 1998).

A double-ring infiltrometer test, with a falling inner head, was performed at the study site at one location, in unsaturated soils to measure the soil infiltration rate. The purpose of the component of the study was to assist in the characterization of the hydrologic conditions within
the study site. No additional statistical analysis was performed on this data. The location of the test occurred on the west side of the Lemes channel, in the lower reaches, within the floodplain (channel left). This site was selected as it is an area of the basin that is known to inundate under high flow conditions. The measurements were used to estimate of the soil conductivity at the site. For this test a double-ring infiltrometer was used with an outer ring diameter of 30.478 cm and the inner ring has a diameter of 15.239 cm. A total of 1-liter of water was used to perform this test. Initially the outer-ring was filled to the top edge, water was kept at a constant head for approximately 10 minutes. The inner-ring was filled after the outer-ring. The inner-ring kept at a constant head for 2 minutes and thirty-three seconds. Change in water depth measurements were recorded every minute, thereafter, for approximately 30 minutes.

The measurements reordered were used to estimate the hydraulic conductivity of the unsaturated soils using Equation 5.

\[
I_t = \frac{\Delta V}{A \Delta t}
\]  
(Equation 5)

Where \(I_t\) is the infiltration rate (length per second); where \(\Delta V\) is the change in water volume of the inner ring (cubic inches) derived from regular measurements of the water height; where \(A\) is the area of the inner-ring (squared inches); and where \(\Delta t\) is the change in time (seconds) between each measurement. The estimated infiltration rate for each of the measurements of the allotted time were used to calculate the average infiltration rate (in units of length per time). For this study the results are presented but no further analysis was performed.
Results

Estimated Peak Discharge Values

Estimated discharge values were calculated using two separate methods. Results are reported in Standard International (SI) units, cubic meters per second ($m^3/s$) with their US Customary Units equivalents listed for reference. A drainage basin area of 12.5 square miles was calculated. The average standard error of prediction reported for both models, reported in Table 4 and Table 5 includes average sampling error and average standard error of regression calculated by the USGS.

Table 4 (Top) Output of the estimated peak discharge values using the USGS Sum of Least-Squares Regression method Table 5 (Bottom), Output of the estimated peak discharge values using the USGS National Streamflow Statistics model (NSS).

<table>
<thead>
<tr>
<th>USGS Generalized Least Squares Regression</th>
<th>ft3/s</th>
<th>m3/s</th>
<th>Standard Error %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Discharge</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q2</td>
<td>460.39</td>
<td>13.04</td>
<td>63</td>
</tr>
<tr>
<td>Q5</td>
<td>904.06</td>
<td>25.60</td>
<td>48</td>
</tr>
<tr>
<td>Q10</td>
<td>1291.48</td>
<td>36.57</td>
<td>41</td>
</tr>
<tr>
<td>Q25</td>
<td>1895.84</td>
<td>53.68</td>
<td>38</td>
</tr>
<tr>
<td>Q50</td>
<td>2436.03</td>
<td>68.98</td>
<td>37</td>
</tr>
<tr>
<td>Q100</td>
<td>3051.98</td>
<td>86.42</td>
<td>38</td>
</tr>
<tr>
<td>Q500</td>
<td>4841.13</td>
<td>137.09</td>
<td>45</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>National Streamflow Statistics</th>
<th>ft3/s</th>
<th>m3/s</th>
<th>Standard Error %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Discharge</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PK2</td>
<td>462</td>
<td>13.08</td>
<td>63</td>
</tr>
<tr>
<td>Pk5</td>
<td>908</td>
<td>25.71</td>
<td>48</td>
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<tr>
<td>PK10</td>
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<td>PK25</td>
<td>1900</td>
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<td>37</td>
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<td>PK100</td>
<td>3060</td>
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</tr>
<tr>
<td>PK500</td>
<td>4860</td>
<td>137.62</td>
<td>45</td>
</tr>
</tbody>
</table>
Statistical Analysis of Model Output

Flow simulation under steady state conditions was performed for ten geometric plans (groups) at the 2-year estimated peak discharge, with a value of 13.04 ($m^3/s$). Population means of the response variables, average total velocity (cross section), and hydraulic depth (cross section), were tested among all groups to test for variance. A baseline plan (no ineffective flow) was compared to nine alternative plans (ineffective flow areas). Manning’s n roughness coefficient was held constant for all ten geometric plans (Manning’s n baseline values used in modeling; LOB .05, C .035, ROB .05).

Informal tests of normality were performed among geometric plans (groups; 1 baseline, 8-16 alternatives) (See Figures 7-10 average velocity; Figures 8-11 hydraulic depth). A One Way Analysis of Variance (ANOVA) was conducted across ten groups of equal sample size (See Table 5 and 6, formal statistical test results).

Analysis of Average Velocity
Figures 11-14 Informal testing of the data for average total velocity among ten selected groups, (Figure 11, top left pg. 30) box and whisker plot, (Figure 12, top right pg. 30) histogram, violin, and box plot, (Figure 13; bottom left pg. 31) bootstrap sampling of the mean, (Figure 14, bottom right pg. 31) QQ Test.

Informal testing of average total velocity data demonstrates the data range and spread of the data are consistent. The data appear to be symmetric and fairly heavy tailed. The QQ (Quantile-Quantile) Test is a test of normality, both the left and right sides point up suggesting the data is heavy tailed. The QQ test suggests that there is some degree of non-normality among the data. A bootstrap sampling of the mean was performed and demonstrates the data is normal.

Results from Shapiro-Wilk and Anderson-Darling formal tests of normality indicate the data is normal. The presence of outliers, more than expected for the sample size was noted. The presence of outliers could be related to the geomorphology of the channel bed, among other things. Shapiro-Wilk and Anderson Darling tests were used as they are more sensitive to dealing with the presence of outliers then other statistical tests.

An ANOVA was performed to test equal populations of the mean. At the .05 level I fail to reject the null hypothesis $H_0 : \mu_1 = \mu_2 = \mu_3 = \mu_4 \ldots \mu_{10}$. There is insufficient evidence to claim differences among population means. I am 95% confident that there are no statistically significant differences in the mean value of the response variable average total velocity among
populations. The variation in the Mean Squares (Mean Sq.) values indicates there is very little variation between the means.

Table 6 Statistical output computed for the response variable average total velocity.

<table>
<thead>
<tr>
<th>Shapiro-Wilk</th>
<th>W = 0.80227, p-value &lt; 2.2e-16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anderson-Darling</td>
<td>A = 37.809, p-value &lt; 2.2e-16</td>
</tr>
</tbody>
</table>

**One Way Analysis of Variance (ANOVA)**

<table>
<thead>
<tr>
<th></th>
<th>Df</th>
<th>Sum Sq</th>
<th>Mean Sq</th>
<th>F value</th>
<th>Pr(&gt;F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>9</td>
<td>0.001</td>
<td>0.000141</td>
<td>0.007</td>
<td>1</td>
</tr>
<tr>
<td>Residuals</td>
<td>500</td>
<td>10.015</td>
<td>0.02031</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Analysis of Hydraulic Depth**

Figures 15-18 Informal testing of the data for hydraulic depth among ten defined groups, (Figure 15, top left) box and whisker plot, (Figure 16, top right) histogram, violin, and box plot, (Figure 17; bottom left) bootstrap sampling of the mean, (Figure 18, bottom right) QQ Test.
Informal testing of hydraulic depth data demonstrates the range and spread of the data is consistent. The data appears to be symmetric and fairly heavy tailed. The QQ (Quantile-Quantile). The QQ test suggests that there is some degree of non-normality among the data. A bootstrap sampling of the mean was performed and demonstrates the data is normal.

Results from Shapiro-Wilk and Anderson-Darling formal tests of normality indicated the data is normal. The presence of outliers, more than expected for the sample size was noted.

An ANOVA was performed to test equal populations of the mean. At the .05 level I fail to reject the null hypothesis $H_0: \mu_1 = \mu_2 = \mu_3 = \mu_4 \ldots = \mu_{10}$. There is insufficient evidence to claim differences among population means of hydraulic depth. I am 95\% confident that there are no statistically significant differences in the mean value of the response variable hydraulic depth among populations. The variation in the Mean Squares (Mean Sq.) values indicates there is very little variation between the means.

Table 7 Statistical output computed for the response variable hydraulic depth.

<table>
<thead>
<tr>
<th>Shapiro-Wilk Normality Test</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$W = 0.9672$, p-value = 3.55e-09</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Anderson-Darling Normality Test</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$A = 6.2943$, p-value &lt; 1.794e-15</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>One Way Analysis of Variance (ANOVA)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Df</td>
</tr>
<tr>
<td>-----------------</td>
<td>----</td>
</tr>
<tr>
<td>Type</td>
<td>9</td>
</tr>
<tr>
<td>Residuals</td>
<td>500</td>
</tr>
</tbody>
</table>

**Sensitivity Analysis**

A sensitivity analysis was performed to test the sensitivity of the one-dimensional model to Manning’s $n$ roughness coefficient by comparing the baseline values of Manning’s $n$ to
± 30%. The sensitivity analysis was performed by globally adjusting values at the ± 10% level. The analyses were performed for the 2-year estimated peak discharge and are reported in terms of the response variables as well as water surface elevation. The results are presented in graphical format (See Figures 19, 20, 21, 22). The results are summarized in table format (See Table 8).

Figure 19 Manning’s n Sensitivity Analysis (± 30 %) results of water surface elevation for an estimated 2-year estimated peak discharge under steady state conditions.
Figure 20 Manning’s n sensitivity analysis results of average velocity for the estimated 2-year estimated peak discharge under steady state conditions.

Figure 21 Manning’s n sensitivity analysis results of hydraulic depth for the estimated 2-year estimated peak discharge under steady state conditions.
Figure 22 Manning’s n sensitivity analysis for average velocity for the estimated 2-year flood under steady state conditions. This illustrative purposes this graph shows the extreme bounds of the sensitivity analysis.

Table 8 A summary of results for the Manning’s n sensitivity analyses

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manning’s n sensitivity analysis for all response variables demonstrated anticipated directional changes</td>
<td>-</td>
</tr>
<tr>
<td>An increase (+) in Manning’s n yielded a decrease in average velocity and an increase in hydraulic depth</td>
<td>-</td>
</tr>
<tr>
<td>A decrease in (-) Manning’s n yielded an increase in average velocity and a decrease in hydraulic depth</td>
<td>-</td>
</tr>
</tbody>
</table>

**Falling Head Permeability Analysis**

A falling head permeability test was performed using a double ring-infiltrometer. Measured changes in water depth within the inner ring of the infiltrometer were taken every 60 seconds for 30 minutes. The test was performed on unsaturated soils in the channel bottom, during mid-afternoon, full sun.
Figure 23 Test results from the falling head permeability test are shown graphically, where length (inches) v. time (seconds) is depicted. The test was performed in the field. Soils conditions were unsaturated. Data were collected from one site. The purpose of this test was to further quantify hydrologic conditions within the study site. No statistical analysis was performed using the collected data.

Table 9 Falling head permeability test results

<table>
<thead>
<tr>
<th>Average Infiltration Rate (in/hr.)</th>
<th>0.054</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Infiltration Rate (in/day)</td>
<td>1.295</td>
</tr>
<tr>
<td>Average Infiltration Rate (cm/hr.)</td>
<td>0.138</td>
</tr>
<tr>
<td>Average Infiltration Rate (cm/day)</td>
<td>3.312</td>
</tr>
</tbody>
</table>

Discussion

This study quantified the magnitude of change, for two localized hydrologic response variables, average velocity and hydraulic depth, respectively. Model conditions were set to an estimated 2-year peak discharge of 13.04 m³/s. The models were not calibrated. The model framework can be described as one-dimensional, steady state and subcritical in nature. The models integrate synthetic ineffective flow areas as a proxy for ‘engineered’ (e.g. constructed log jams) at the reach scale. The cross sections in the model were defined using collected data, raster data,
topographic data, and interpolated data. A slope of .023 (m/m) was calculated from the collected cross section data. A falling head permeability test was performed to help further characterize the hydrologic conditions within the study site.

This study has been described in the context of the current regulatory framework. The current regulatory framework for performing restoration activities in ungaged ephemeral and intermittent streams can be described as layered and occurring at different scales. The regulatory framework encompasses both federal and state agencies. The institutional arrangements comprising the regulatory framework include both federal common law and state statute. In New Mexico, compliance under the Nationwide Permit system and with state water quality standards are enforced and certified by the New Mexico Environment Department and the United States Army Corps of Engineers.

Sensitivity analyses of Manning’s n were performed using baseline conditions (zero ineffective flow areas). For these analyses Manning’s n was adjusted globally and the range of analysis was ± 30% from the baseline estimated values. The results from the Manning’s n sensitivity analyses for both response variables demonstrated anticipated directional changes wherein the rate of change for among the response variable ranged from approximate ± 5-15%. The results from the Manning’s n sensitivity analyses suggest that the model is working properly. As the roughness parameter was increased globally, decreases in average total velocity, and increases in hydraulic depth and water surface elevation occurred. The model sensitivity demonstrates that some directional changes in the response variable can be achieved under the imposed modeling conditions using synthetic ineffective flow areas.

The formal statistical test results clearly indicate there are no statistically significant differences in mean values with respect to the two response variables among all ten populations.
(groups) considered. The results suggest there is no evidence that ineffective flow areas, as a proxy for log jams, are effective at decreasing the average velocity or increasing the hydraulic depth. Based on the statistical evidence there is no reason to believe that the addition of engineered log jams at the reach scale is an effective restoration technique.

The results identify that restoration-derived enhancement to average velocity and hydraulic mean may be difficult to measure and even harder to demonstrate at the reach scale. This result is in line with current literature which suggests larger reaches need to be analyzed and under different flow conditions, for example, reaches as much as 5-10 km may need to be modeled in order to produce enhancements that would justify channel restoration (Sholtes and Doyle, 2011). Given the small extent of most restoration projects in the United States (1 km, Bernhardt et al.2007), and the small size of channels generally restored first to third order, a question of restoration scale and practice is raised.

It is not clear what type of impact performing a non-steady state simulation, under mixed flow conditions would have on the modeled reaches created in this study. However one study (Sholtes and Doyle, 2011) suggests that modeling under non-steady conditions using both field based and synthetic reaches, restoration-derived enhancements, such as wood debris, to floodwave attenuation, is also difficult to measure and also demonstrate. The research reported in Sholtes and Doyle (2011), using dynamic flood routing described channel restoration itself, as currently practiced, is insufficient to provide significant hydrologic changes.

One-dimensional flood routing models are widely applied to practical and theoretical questions associated with flood wave routing (Knight, 2005) however they are not without limitations. For example, they do not explicitly account for all dimensions of flow or in other words they lack multi-dimensional sophistication. As described by Shiono (1999) and Knight and
Shiono (1996) they do not explicitly account for two and three-dimensional aspects of energy dissipation due to turbulence exchange at the interface of floodplain and channel flows and momentum lost in transverse flows around meander bends. Similarly, this model does not accurately capture two and three-dimensional aspects of changes in velocity around ineffective flow areas.

There are limitations to this study. The study does not account for multi-dimensional flows, mixed flows, the model is not calibrated, Manning’s n values are defined laterally but not vertically, Manning’s n values are static during simulated runs, and entrapped natural woody debris is not accounted for. Running simulations where multi-dimensional flows and other flow regimes, such as mixed flow, are considered would certainly enhance the model output and improve the robustness of the simulated runs, but to what extent remains unclear.

It was observed that the Froude values were near or approached the value of one at many cross sections, under baseline conditions, as well as, plans that impose hypothetical conditions. Specifically, Froude values reported of one or greater for more than one of the cross sections across all ten compared groups were observed. This observation suggests that the flow also is subcritical and critical in nature within the channel. Froude value is a dimensionless parameter measuring the ratio of the inertia force on an element of fluid. The Froude number is hydraulically relevant because the weight of the fluid and the gravitational forces acting upon water in the channel can have an impact on the modeling results.

Calibrating the model to historical flow data or high water marks would improve the robustness and reliability of the model. There is currently no available historic flow data for Lemes Channel. No high water marks were observed in the field. Defining Manning’s n vertically among all cross sections in the model would also improve the robustness of the model.
by integrating more reality into the model framework. Furthermore, anytime the real world is modeled there are uncertainties. Merwede et al. (2008) discussed uncertainties in hydrologic models including topographic representation, precipitation, estimated peak discharges, as well as limitations with respect to one-dimensional steady state hydraulic modeling (Merwede et al., 2008). This model study is also limited in that it does not account for sediment transport/erosion, bulking factors.

The assumptions of this model (i.e. one-dimensional, steady state, subcritical) do influence the modeling results but do not necessarily make the modeling results irrelevant. For example, one-dimensional flows are considered applicable to natural open channel flow for small streams, and generally provide accurate results or predictions for two-dimensional flow when there are few meanders, flow is completely smooth, and subcritical (Hubbard, 2001).

The integration of synthetic ineffective flow areas are meant to be representative of the hydrologic processes that could occur under certain types of field based conditions considering various restoration activities. While there are assumptions as well as limitations to this model, one-dimensional models are used to simulate natural open channel flows. The statistical results identify the importance of hydrologic design elements used in channel reconfiguration projects. More research is needed to understand how ineffective flow areas (i.e. engineered log jams) within ephemeral and intermittent channels in arid and semi-arid climates affect average total velocity and hydraulic depth under other flow conditions (i.e. critical, supercritical, mixed flow) and peak discharges at the reach scale. While there is anecdotal evidence, not presented here, within Lemes Channel that under localized conditions desired outcomes such as dispersal of flood wave energy, improved vegetation, and floodplain inundation can occur there are no results
presented in this study which suggest that this would be an effective technique for decreasing average velocity or increasing hydraulic depth.
Acknowledgments

I’d like to acknowledge Bob Berrens and the Water Resources Program at UNM for providing an excellent learning environment and opportunities. I’d like to thank my adviser Mark Stone for providing exceptional guidance and academic mentorship. Bruce Thomson and Vince Tidwell provided much appreciated research critique and assistance. I’d like to acknowledge the committee’s role in helping shape the research goals of this project as well as the scope and direction. The committee provided comments and editorial suggestions that were integrated into this work. Thanks to Connie Maxwell, Richard Davidson and the Alamosa Land Institute, Colin Byrne, Dadhi Adhikari, Ryan Kelley, Reed Benson, Adrian Oglesby and Neil Schaffer. A special thanks to David Brookshire and the Department of Economics for hiring me on as a Research Assistant and providing me with financial assistance. Lastly, thanks to my wonderful family who helped make this possible. Special thanks to my wife Amy, kids Georgie and River for providing endless support and laughter throughout.
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Alamosa Land Institute, 2011, Canada Alamosa Watershed Restoration Action Plan


