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A MORPHOLOGICAL AND ECONOMIC EXAMINATION OF PLUNGE POOLS AS ENERGY DISSIPATERS IN URBAN STREAM CHANNELS[1]

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ABSTRACT: Naturally formed plunge pools (scour holes) are a common morphologic feature in many urban stream systems where the transition between a pipe and a natural channel occurs. Plunge pools serve as significant stream energy dissipaters, increasing flow resistance and enhancing stream channel stability. Such features may also improve habitat diversity and serve as refugia for stream biota during low flow periods. The morphologic characteristics of several naturally formed plunge pools associated with road crossing culvert outlets in the metropolitan Charlotte, North Carolina, area are presented. Plunge pool dimensions surveyed include maximum depth, length, and width, and longitudinal and side slopes as well as bed material. Culvert outlet dimensions and hydraulic characteristics of the scouring jet for each study site are also reported. Design equations developed from flume studies generally failed to predict the naturally formed plunge pool dimensions. Pool volume was significantly correlated with drainage area, with pool depth being the least sensitive dimension to changes in the magnitude of the scouring flow. The excavation costs for designed plunge pools compare favorably to initial construction costs of traditional culvert outlet riprap aprons.
(KEY TERMS: surface water; hydrodynamics; plunge pool; scour; culvert; urban stream morphology; geomorphology.)

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INTRODUCTION

In 1997 the City of Charlotte began investigating the implications of its current maintenance practices for urban channels at road culvert crossings. The investigations were initiated after it became apparent that certain aspects of these practices could destabilize downstream channels and accelerate erosion, reinitiating a larger cycle of instability in an otherwise stable "built-out" watershed. Increasing urbanization upstream of many culvert crossings has resulted in the erosion of the original armoring riprap apron, as erosive flows have become more frequent and intense. The long term maintenance of degrading riprap aprons at culvert crossings is expensive, and the resultant crossing instability often impacts water quality and erodes streambanks on public and private property. Culvert outlets that have not been modified in more than 10 years and have become adapted to the urban watershed display geomorphic characteristics of stability. Historic surveys at culvert outlets (Keeley, 1963; A.C. Scheer, 1968, unpublished report, University of Montana, Missoula, Montana) identified both the presence of gully scour that migrated upstream or localized scour holes below road culvert crossings. Gully scour was usually associated with unstable downstream channel slopes, while localized scour was associated with more stable downstream channel slopes. The development of an apparently stable plunge pool. is a recurring theme at culvert outlets underlying urban road crossings where the periodicity of the maintenance of downstream riprap or other channel armoring is infrequent. Often when such pools are simply filled in with more riprap, a common maintenance practice, significant localized bank erosion and channel down cutting has resulted.

The prediction of scour geometry at culvert and cantilever pipe outlets is a regular component of culvert or dam design methodology in determining the need for erosion protection. Federal Highway Administration procedures (FHWA, 1983) present outlet and

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scour geometry relationships for scour depth, width, length, and volume as a function of discharge intensity ($Q/D^{2.5}$) where Q is the discharge in cfs and D is the pipe diameter in feet. Similar relationships were proposed earlier by researchers from the U.S. Army Corp of Engineers (USACE) (Bohan, 1970; Fletcher and Grace, 1972). Modified relationships to account for variations in culvert shape are presented by Abt *et al.* (1987). U.S. Department of Agriculture (USDA) researchers have developed design criteria for pipe spillway outlets suitable for farm pond outlets or flood control reservoirs.

Initially, plunge pool dimensions were determined for free flowing cantilevered pipe outlets (USDA, 1986; Blaisdell and Anderson, 1991). These design criteria were later modified for fully submerged (Rice and Kadavy, 1994) and partially submerged pipe outlets (Rice and Kadavy, 1995). Several limitations exist in the research carried out to date. In all instances, model flume studies were used to predict scour geometry with little or no field measurements. Such studies cannot account for the influence of vegetation and streambed and bank material heterogeneity on scour hole geometry. The influence of multiple culvert openings on scour geometry, a relatively common feature in urban settings, has not been investigated. Most of the above investigations primarily examined channel scour in noncohesive sediments; only the FHWA investigators analyzed scour in cohesive sediments (Ruff *et al.,* 1982); and only Blaisdell and Anderson (1991) considered the effect of impeding layers on scour geometry. Perhaps most importantly, the scour geometries generated by these different methodologies have not been compared with each other or with examples of localized channel scour in the field.

Armored culvert outlet aprons designed to protect the road crossing and prevent downstream erosion may consist of costly, elaborate concrete structures or riprap channel lining sized to the culvert opening and design discharges. Failure of armored culvert outlets can be attributed to any or all of these conditions: incorrect installation, lack of maintenance, and discharges that exceed initial design specifications. Ultimate scour dimensions will be determined by the erosive power of the stream, channel materials, streambank vegetation, and pipe outlet or tailwater geometry. The use of preformed plunge pools below culvert openings as an energy dissipation structure may represent a viable alternative to riprap channel linings or other structures in settings where the downstream channel slope is stable. Plunge pools are a commonly used energy dissipation structure for dams of all sizes (Hager, 1998), and riprap lined outlet basins are a recommended design feature for culvert crossings (FHWA, 1983). Plunge pools may prove to be inherently more stable than other structures in a developing urban environment, thereby reducing long term maintenance costs. Also plunge pools are preferable to riprap channel linings in natural channel design stream restoration projects, providing stream habitat diversity and in some instances stream aeration.

Here the geometry of several plunge pools that have formed naturally below road culverts as a result of riprap channel lining failure in the Charlotte metropolitan area is characterized. Natural plunge pool dimensions are compared to the culvert scour geometry predicted by FHWA, USACE, and the USDA design equations. The costs associated with plunge pool excavation are compared to those for initial construction of traditional riprap outlet aprons currently required under present design standards.

METHODOLOGY

A field survey of several naturally formed plunge pools was conducted in the Charlotte metropolitan area. The pools were surveyed with a stadia rod and theodolite or by chaining the dimensions of each pool. Plunge pool survey dimensions were collected by wading the plunge pool or from a small boat in larger plunge pools. Culvert dimensions and outlet configurations were also recorded at this time. Streamside gauge boards were installed at the distal end of most plunge pools and read manually during high flow conditions to establish the relationship between tailwater and culvert elevations. Sediment samples were collected from the streambed and banks and where present from the outlet bar at the downstream end of each plunge pool. Samples were analyzed at the University of North Carolina (UNC)-Charlotte sedimentology laboratory for grain size distribution.

Sediment samples were first dried at 105°C for 24 hours prior to analysis. The bulk sediment sample was then weighed and sieved through progressively smaller screens at 0.5ϕ intervals until only the $< 4\phi$ fraction remained. The $< 4\phi$ fraction was then analyzed with a Spectrex Laser Particle Counter, and those results were combined with sieve results to determine the complete grain size distribution for each sample.

An extensive literature review was conducted to explore the use of plunge pools as energy dissipaters in fluvial systems and to investigate plunge pool design criteria for different hydraulic settings. In addition, storm water and engineering utilities of major urban centers throughout the United States were also contacted for information regarding the use of plunge pools as energy dissipation structures. Scour prediction equations developed by the USDA

(USDA, 1986; Blaisdell and Anderson, 1991; Rice and Kadavy, 1994; Rice and Kadavy, 1995), the FHWA (Ruff *et al.,* 1982), and the USACE (Bohan, 1970; Fletcher and Grace, 1972) were identified as commonly used methodologies to predict pipe or culvert outlet scour.

Of the three scour prediction methodologies, the USDA approach requires the most input variables; these include culvert diameter (D_o), culvert elevation, discharge, mean pipe velocity, tailwater elevation, culvert slope, mean sediment grain size (D_{50}), and elevation of the natural streambed. Attempts to utilize the USDA methodology to predict the dimensions of the naturally formed plunge pools in the study dataset were unsuccessful. Utilizing only two-year return interval discharges in the USDA Design Note Six methodology generated pool dimensions far larger than any in the study dataset and/or violated the pool stability criteria used in the methodology (Blaisdell and Anderson, 1988a). The small D_{50} values found in these study streams are much smaller than the armored bed material used to develop the series of predictive equations presented in USDA (1986), Blaisdell and Anderson (1991), and Rice and Kadavy (1994, 1995). The designers stated their intent that the USDA methodology should be used to size armored riprap lined stilling basins rather than unlined plunge pools scoured in fine grained streambeds.

The general expression used in the FHWA methodology to predict dimensionless scour geometry in cohesionless material for a circular culvert is

$$h_s/y_e,\ w_s/y_e,\ L_s/y_e \text{ or } V_s/y_e = \alpha_e(Q/\sqrt{g}\ y_e^{2.5})^B(t/t_o)^\Theta \quad (1)$$

where h_s, w_s, L_s, and V_s are the depth, width, length (all in ft), and volume (ft^3) of scour, respectively; Q is the discharge (ft^3/sec); g is the acceleration of gravity (ft/sec/sec); t is the time (min); t_o is a base time (316 minutes unless specified otherwise) used to derive the empirical coefficients α, B, and Θ; and y_e is the depth of flow (ft) that is equal to the culvert diameter D (ft) or $(A/2)^{0.5}$ for noncircular or partially full culverts with A being the cross sectional area of flow (ft^2).

Where the duration of the erosive flow (t) is unknown, a value of 30 minutes is typically used for design purposes where observations as to the duration of peak flows are unknown (FHWA, 1983). The FHWA methodology also considers scour in cohesive sediments using the expression

$$h_s/y_e,\ w_s/y_e,\ L_s/y_e \text{ or } V_s/y_e = \alpha_e(\rho v^2/\tau_c)^B(t/t_o)^\Theta \quad (2)$$

where $\rho v^2/\tau_c$ is the modified shear number (ft/sec/sec), v is outlet mean velocity (ft/sec), and τ_c is the critical tractive shear stress (lb/ft^2) that is defined as

$$\tau_c = 0.0001\ (S_v + 180) \tan (30 + 1.73\ PI)$$

where S_v is the saturated shear strength in pounds per square inch, PI is the plasticity index from the Atterberg Limits, and ρ is the fluid density (lb/ft^3). The recommended application of Equation (2) is limited to sandy clay soils with a plasticity index between 5 and 16. Values for the empirical coefficients α, B, Θ are provided for cohesive and cohesionless soils and for uniform and graded material as well as for culverts other than circular (FHWA, 1983).

A similar series of empirical relationships is presented by the USACE (Fletcher and Grace, 1972). The general forms of the relationships are

$$D_{sm},\ W_{sm},\ L_{sm}/D_o = a\ (Q/D_o^{2.5})^b\ t^c \quad (3)$$

or

$$V_{ss}/D_o^3 = a\ (Q/D_o^{2.5})^2\ t^c \quad (4)$$

where D_{sm}, W_{sm}, L_{sm} and V_{ss} are the depth of scour, one-half the total width of scour, the length of scour (all in ft), and the volume of scour (ft^3); D_o is the culvert diameter (ft); and a, b, and c are empirical coefficients.

Fletcher and Grace (1972) also present alternate empirical coefficients for tailwater elevations above and below 0.5 D_o. However, cohesive sediments were not considered in the derivation of this methodology, and only gravel sized material was used in the original flume experiments.

The scour prediction equations presented by the USACE and the FHWA require the input of a design discharge to predict scour geometry. In this study the final scour dimensions are actually known, but the discharges that formed the various pools are unknown. Most of the plunge pools examined in this study were situated on ungauged stream sections in the Charlotte area. Historic observations of the ages of the various pools is lacking, but maintenance records indicate that the ages of all pools examined in this study as more than 10 years (City of Charlotte Stormwater Maps). Two extreme flood events (August 1995 and July 1997) occurred in the Charlotte and Mecklenburg County area in the past 10 years (USGS, 1998). An examination of the spatial distribution of rainfall totals during these two events indicates that all but two of the pools – Lawton Road and Lawton Road railroad crossing (RR) – were impacted by erosive flows generated by rainfall events with a return interval of more than 100 years in either or both events. The Lawton Road and Lawton Road RR pools saw flows generated by rainfall totals with a return interval of more than 25 years but less than 50 years. Flume tests have indicated that approximately two-thirds to three-fourths of the maximum scour occurs in the first 30 minutes of the peak flow duration (FHWA, 1983). Therefore, the pool geometries measured in this study have formed under a flow regime greater than the 25-year return interval required for most road crossing culvert outlet designs. An estimate of the discharges necessary to scour the observed plunge pool dimensions using the FWHA and USACE methodologies was determined by back calculating Equations (1), (3), and (4). Equation (2) was not used in this study, as bed and bank material samples from the various pools were considered to be noncohesive material. USACE HEC-1 software V. 4.0 was used to generate the 2-, 10-, 25-, 50-, and 100-year return interval flows for each of the plunge pool locations. Return intervals of the precipitation inputs that drive the HEC model runs are those used by Charlotte Stormwater Services and are based on historic precipitation records for the Charlotte area. Soil Conservation Service (SCS) curve numbers (CN) for the watershed areas above each culvert outlet were calculated from Charlotte's 1998 digital land use coverage and tax parcel information using ArcView geographic information system software. The flow paths used to estimate the time of concentration values for each plunge pool location were determined from City of Charlotte engineering maps and aerial photos and where necessary field investigations to determine storm water sewer connections and pipe sizing. The methodology used to estimate the 2-, 10-, 25-, 50-, and 100-year discharges followed standard culvert sizing and design practices used by Charlotte Stormwater Services. Exponential regression equations were generated from the 2-, 10-, 25-, 50-, and 100-year discharge estimates for each station and used to estimate the return intervals of the pool forming discharges determined from the USACE and FWHA design methodologies.

Of particular interest to storm water and engineering utilities is the potential initial and long term cost savings of utilizing designed plunge pools in place of traditional riprap culvert aprons. The cost to excavate the naturally formed pools observed in this study was compared to the initial construction cost of riprap culvert aprons. Dimensions of culvert outlet riprap aprons were estimated using the City of Charlotte's culvert engineering erosion control guidelines. Discharges with a 25-year return interval were used to size the outlet apron. Costs associated with the construction of culvert outlet aprons include excavation (\$15.70 per m^3), riprap (\$26.45/ton), and Coco Matting (\$2.50/m^2). Excavation costs for plunge pool construction were estimated as \$15.70 per cubic meter of material removed.

RESULTS

A summary of the morphometric and hydraulic features of the plunge pools examined in this study is presented in Table 1. Side and top view schematic diagrams defining the plunge pool dimensions are given in Figures 1a and 1b, respectively. The drainage area above the various pools ranged from 0.07 km^2 to 2.03 km^2. The land use of most pools was primarily urban with SCS CN's raging from 71 to 95. Culvert diameters varied from 0.46 m to 2.3 meters, and all except Lawton Road RR were circular in shape. The erosive jet for 7 of the 18 pools was supported by the downstream pool, meaning the culvert opening was at least partially submerged during high flow conditions (Figure 1a). The scouring jet for the remainder of the pools exhibited a free fall configuration even under high flow conditions. Seven of the pools were fed by more than one culvert, and exposed bedrock or large debris was found on the bottom of five of the pools. Pools with impeding bedrock bases were excluded from any analyses that required the depth of scour or pool volume as input variables. The stability of two pools – Belvedere Avenue and Cove Creek Drive – was questionable, owing to recent maintenance activities; thus they also were excluded from most analyses.

Despite the drainage area varying over three orders of magnitude, the coefficient of variation for the major pool dimensions – maximum depth of scour (Z_{max}), maximum width of scour (W_{Tw}), and maximum length of scour (L_t) – averaged only 34 percent (Table 1). The dimensions of the area of maximum scour (pool bottom area, L_u, L_s, W_s) and the pool volume (V) varied more significantly. As might be expected, a significant positive relationship existed between drainage basin area above the culvert crossing and plunge pool volume (Figure 2a). A similar positive relationship is evident when discharge and pool volume (Figure 2b) are compared. Lower correlations are found between discharge and individual pool dimensions (Z_{max}, W_{Tw}, and L_t) (Figures 3a, b, and c).

A summary of the grain size characteristics of the streambed and bank and outlet bar sediments is presented in Table 2. The bed and bank deposits were generally dominated by noncohesive sediments, with the median grain size (D_{50}) for most pools classified as fine grained to coarse grained sand (Dyer, 1986). The Cove Creek Drive bed material was skewed to a coarser grain size fraction by the presence of riprap fragments. The cohesive grain size fractions averaged 10 to 12 percent of each sample by weight. An outlet bar that occupied almost the entire width of the pool outlet characterized several of the pools (Figure 1b). Similar depositional features were reported to form during flume experiments (Blaisdell and Anderson,

TABLE 1. Natural Plunge Pool Morphology and Hydraulic Characteristics.

	Drainage Area (km^2)	Curve Number	D_o (m)	T_W (m)	Z (m)	Z_{max} (m)	W_{Tw} (m)	L_t (m)	V (m^3)
Independence High School	0.05	76	0.61, 0.46	1.03	1.34	0.85	4.45	6.89	8.6
Belvedere Ave.	1.05	71	1.22	-0.01	1.52	1.00	6.40	10.06	35.2
Cove Creek Drive	0.44	76	1.52	0.22	0.43	1.31	3.43	3.51	NA
Manchester Lane	0.09	82	1.22	0.02	0.44	0.79	5.12	7.38	9.4
Monroe Rd.	0.69	80	1.52,1.52,0.61		1.13	1.37	8.56	9.66	26.1
Carmel Forest	0.12	77	0.61	0.16	0.31	0.78	2.90	7.56	5.9
Lawton Road RR	2.06	82	(1.71x1.89) x 2*	0.40	1.05	1.49	11.43	12.38	77.8
Lawton Road	1.77	82	1.83,1.83	0.56	0.49	1.34	8.29	12.95	55.7
Thermal Road	1.39	72	2.13	-0.13	0.28	0.65	7.53	12.19	28.2
Ranch Road	0.85	70	1.83,0.61	-0.07	0.00	0.80	6.10	17.15	19.0
Ashley Road 1	0.53	84	1.07	-0.23	0.00	1.52	5.49	10.36	33.7
Ashley Road 2	0.56	85	1.07		0.43	0.46	5.97	9.14	8.8
Ashley Road 3	0.59	85	1.07	-0.23	0.00	1.03	7.00	13.91	28.8
Dorn Circle	0.94	81	1.22,0.61	0.29,0.20**	0.37,0.28**	0.88	6.71	9.13	7.3
Orchard Circle	0.53	87	1.37	0.00	0.14	0.93	7.32	13.41	38.7
Eastway Drive	0.70	79	1.37	0.32	0.56	1.83	10.67	11.58	54.6
Independence Blvd.	0.04	93	0.91	0.35	0.36	1.36	5.94	7.98	19.0
Linda Lake Drive	0.44	72	1.52, 0.46, 0.46	-0.06	0.06	0.88	7.01	13.87	27.1
Average	0.71	79.67		0.16	0.50	1.07	6.68	10.51	28.45
Standard Deviation	0.57	6.16		0.34	0.48	0.36	2.18	3.26	19.88
Coefficient of Variation	79.48	7.74		217.92	95.06	33.61	32.68	31.02	69.88
Maximum	2.06	93.00		1.03	1.52	1.83	11.43	17.15	77.77
Minimum	0.04	70.00		-0.23	0.00	0.46	2.90	3.51	5.85

*Two elliptical shaped culverts.
**Culverts at different elevations.
Notes: D_o = culvert diameter, T_w = tailwater elevation, Z = culvert height above streambed, Z_{max} = maximum depth of scour, W_{TW}= maximum width of scour, L_t = maximum length of scour, V = volume of scour.

1988a). The outlet bar deposits consisted of much coarser material than the pool or bank sediments. These were classed as very coarse sands to pebble sized material (Table 2).

The estimated flow return intervals necessary to scour the observed pool dimensions using the FHWA and USACE methodologies in combination with the HEC-1 modeling is presented in Tables 3 and 4, respectively. An examination of the return intervals associated with the calculated PFD reveals that the return intervals of the estimated pool forming discharges (PFDs) for the USACE equations averaged 1.30 years as compared to 3.65 years for the FHWA equations. Despite most of the study sites experiencing discharges from at least one rainstorm greater than a 100-year return interval, the maximum flow return interval required to generate the observed natural pool dimensions was approximately 17 years using the FHWA methodology (Table 3). The seemingly modest scour dimensions measured in this study at least in part reflect the stabilizing influence of streambank vegetation that could not be considered in the flume studies used to develop the scour prediction equations. The relatively high coefficient of variation of the PFD return intervals predicted by both methodologies is attributed to variation among sites in one or more of the following variables: discharge history; the number and variety of sizes and elevations of inlet culverts, bed and bank material composition, and stratigraphy; streambank vegetation; the presence or absence of near surface bedrock; and undocumented construction and maintenance activities.

A summary of the excavation costs necessary to accommodate the natural plunge pools surveyed in this study in comparison to the initial construction costs of traditionally employed riprap culvert outlet

TABLE 1. Natural Plunge Pool Morphology and Hydraulic Characteristics (cont'd).

	L_u (m)	L_s (m)	W_s (m)	Upstream Slope	Downstream Slope	Side Slope	Comments
Independence High School	3.05	3.96	0.58	0.28	0.29	0.11	Free Fall Jet
Belvedere Ave.	1.22	1.83	1.22	0.82	0.12	0.10	Supported Jet
Cove Creek Drive	1.52	1.83	1.83	0.86	0.78	0.41	Free Fall Jet
Manchester Lane	2.13	4.57	1.51	0.37	0.28	0.11	Supported Jet
Monroe Rd.	2.90	5.79	1.19	0.47	0.35	0.09	Free Fall Jet, Debris
Carmel Forest	3.41	4.27	0.52	0.23	0.24	0.16	Free Fall Jet
Lawton Road RR	2.76	4.95	2.38	0.54	0.20	0.08	Free Fall Jet
Lawton Road	2.29	3.62	4.38	0.58	0.14	0.17	Free Fall Jet
Thermal Road	1.52	7.05	0.62	0.42	0.13	0.05	Supported Jet, Bedrock
Ranch Road	4.00	5.52	1.19	0.20	0.07	0.08	Supported Jet, Debris
Ashely Road 1	3.43	7.05	0.91	0.44	0.46	0.17	Supported Jet
Ashely Road 2	1.83	7.25	0.52	0.25	0.24	0.04	Free Fall Jet, Debris
Ashely Road 3	3.43	7.05	0.91	0.30	0.15	0.08	Free Fall Jet, Debris
Dorn Circle	1.52	1.83	0.61	0.58	0.12	0.07	Free Fall Jet, Bedrock
Orchard Circle	1.22	5.79	0.91	0.76	0.12	0.07	Supported Jet
Eastway Drive	4.57	7.01	2.13	0.40	0.40	0.11	Free Fall Jet
Independence Blvd.	1.37	2.59	0.91	0.99	0.25	0.14	Free Fall Jet
Linda Lake Drive	5.79	4.57	0.91	0.15	0.09	0.07	Supported Jet
Average	2.66	4.81	1.29	0.48	0.25	0.12	
Standard Deviation	1.28	1.92	0.94	0.24	0.17	0.08	
Coefficient of Variation	48.17	39.85	73.17	50.97	70.07	69.92	
Maximum	5.79	7.25	4.38	0.99	0.78	0.41	
Minimum	1.22	1.83	0.52	0.15	0.07	0.04	

Notes: L_u = length from culvert to upstream point of maximum scour, L_s = length from downstream point of maximum scour to end of pool, W_s = width of pool bottom at maximum depth.

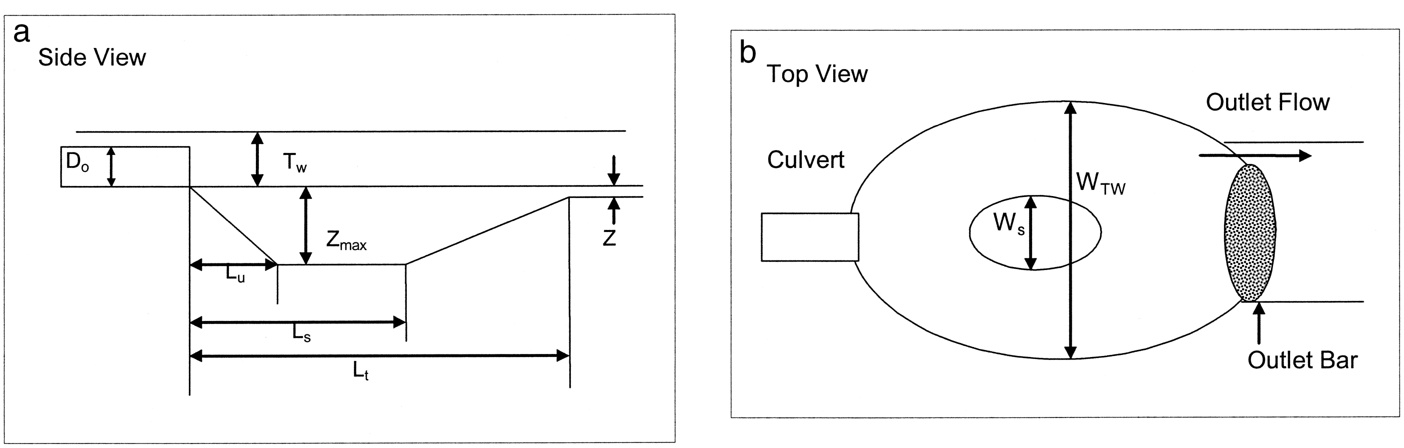
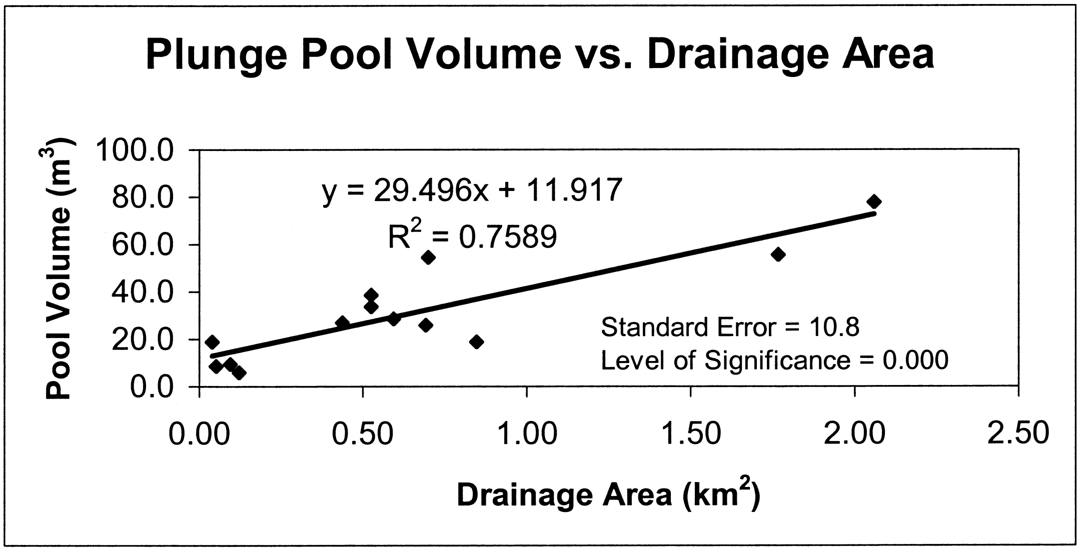


Figure 1. Definition Sketch of Plunge Pool Dimensions: (a) Side View and (b) Top View.



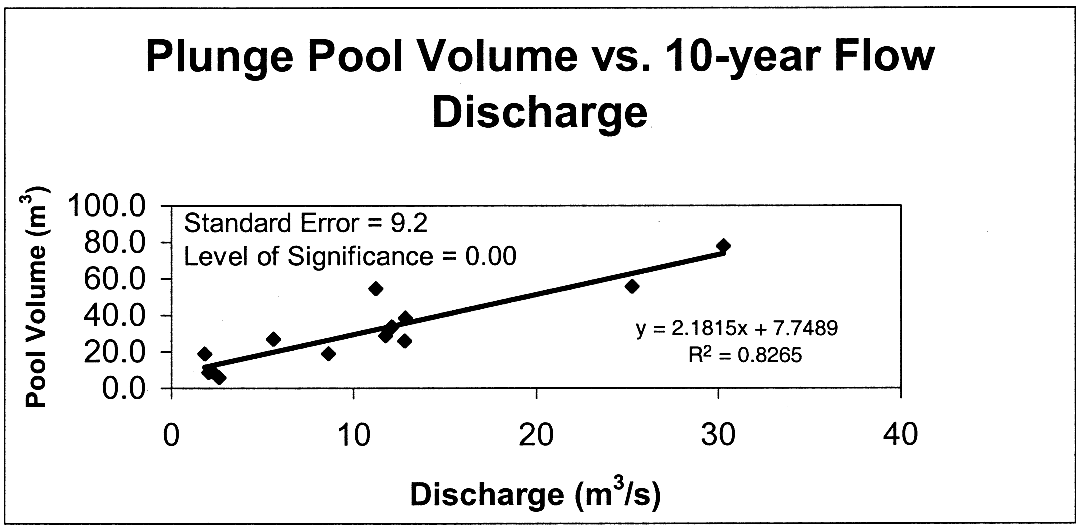


Figure 2. (a) Relationship Between Plunge Pool Volume and Drainage Area and (b) Relationship Between Plunge Pool Volume and Discharge.

aprons is presented in Table 5. In all but the sites with the three smallest plunge pools, substantial initial construction cost savings are predicted. Excluding the three smallest pool sites, excavation costs averaged less than 28 percent of the cost of traditional riprap aprons.

DISCUSSION AND CONCLUSION

The absence of discharge records for the study streams precludes the determination of a maximum discharge that scoured each plunge pool. Therefore, a direct assessment of the FHWA and USACE scour methodologies' ability to predict the natural plunge pool geometries measured in this study is not possible. However, it is known that all but two of the plunge pools analyzed in this study were impacted by discharges generated by at least one precipitation event with at least a 100-year return interval. By using the natural pool and culvert dimensions it is possible to back solve the USACE and FHWA empirical relationships in Equations (1), (3), and (4) to estimate the PFD (Tables 3 and 4). Despite the uncertainties associated with the HEC-1 modeling exercise and the estimation of the exact return interval associated with a particular discharge for each plunge pool location, the consistently short return intervals generated by this analysis indicate that the

FHWA and the USACE methodologies overestimate the scour dimensions that develop in these urban Piedmont streams.

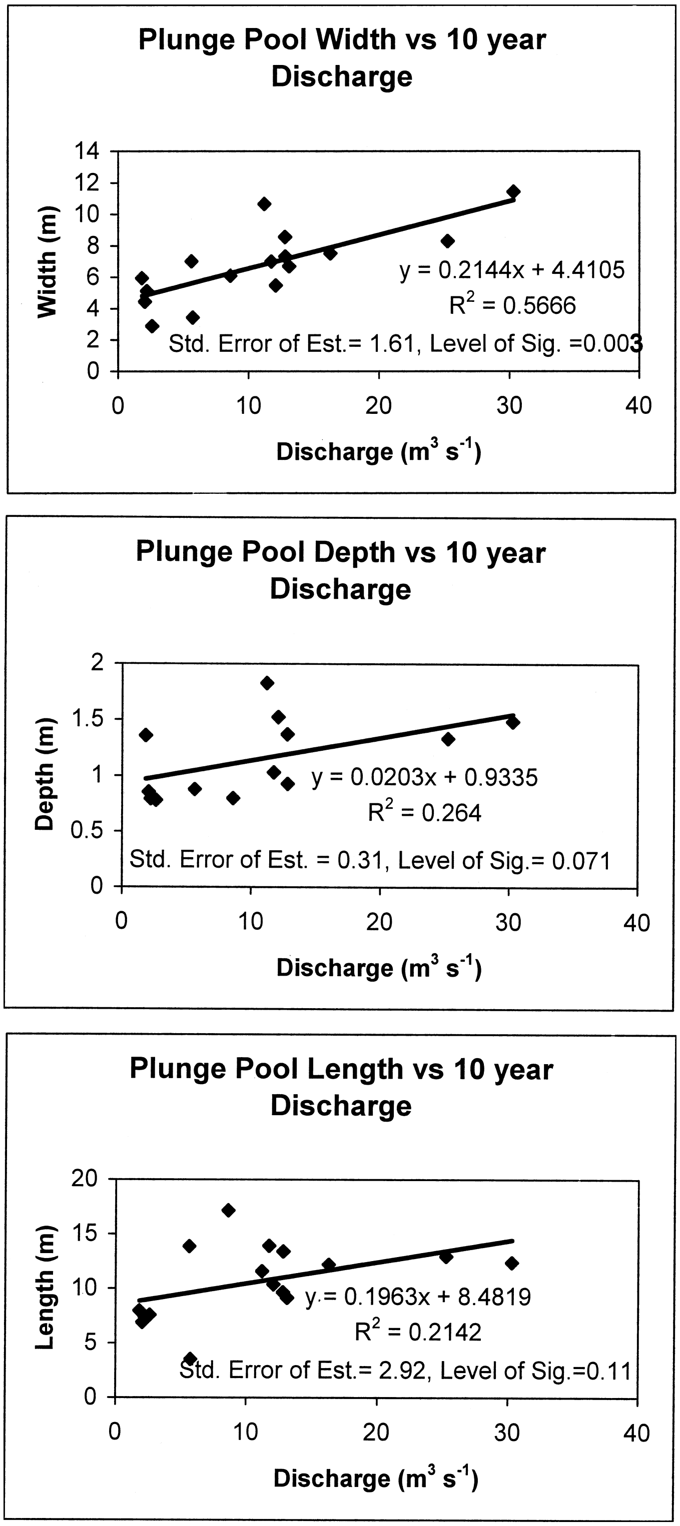


Figure 3. Relationships Between W_{TW}, Z_{max}, L_t and Discharge.

At present the population of surveyed naturally scoured plunge pools is seemingly too small and their dimensions too variable to recommend specific design criteria to predict scour dimensions using either empirical methodology. Relatively poor correlations exist between estimated discharge and pool width, depth, and length (Figure 3). However, higher correlations exist between discharge and drainage area when regressed against plunge pool volume (Figure 2). A series of dimensionless ratios relating the length of scour (L_{Tw}), width of scour (W_{Tw}), and volume of scour (V) to the maximum depth of scour (V_{max}) were developed in a series of flume experiments by Opie (1967). These experiments were carried out with gravel sized material with free falling (NT) and supported tailwater (T) conditions. The average dimensionless geometry (W_{Tw}/V_{max} and $V^{1/3}/V_{max}$) of the natural plunge pools examined in this study is similar to that reported by Opie (1967) (Table 6). However, the L_{Tw}/V_{max} ratio for several of the Charlotte area pools (up to 21) is significantly greater than the ratios of six to eight measured by Opie (1967).

The results from the initial survey of naturally formed plunge pools in the Charlotte area indicate the need for a more extensive field survey of plunge pool geometry. The best correlation from these early results appears to be between drainage area and discharge with plunge pool volume. Given the relatively low variation of Z_{max} and its lower sensitivity to discharge as indicated by the low slope of the relationship depicted in Figure 3, it would seem prudent to set this dimension with consideration towards public safety issues at a specific location and size the other pool dimensions (W_{Tw} and L_t) to match the observed relationship between drainage area and pool volume (Figure 2a). The use of a plunge pool rather than a traditional riprap apron at any culvert crossing must be weighed against the public safety risk presented by such structures, particularly in residential neighborhoods. The Erodibility Index approach developed by Annandale (1995) represents a methodology that might be used to assess the stability of initial plunge pool designs.

A comparison of the costs necessary to excavate the study plunge pools in relation to the construction costs for the installation of armored culvert outlet aprons for these same sites is presented in Table 5. Besides the initial construction cost savings, properly designed plunge pools are inherently more stable than armored culvert outlets, thereby reducing long term maintenance costs. The use of preformed plunge pools is also likely to provide ecological benefits in addition to cost savings by providing habitat diversity and low flow refugia for aquatic biota. Finally, the use of plunge pools as energy dissipation structures as opposed to simply armoring culvert outlets is also

TABLE 2. Grain Size Analysis From Urban Plunge Pools.

	Pool Bed D_{50} (mm)	Percent Fines <0.063 (mm)	Classification	Pool Bank D_{50} (mm)	Percent Fines <0.063 (mm)	Classification	Outlet Bar D_{50} (mm)	Percent Fines <0.063 (mm)	Classification
Independence High School	0.35	12.99	medium sand	0.19	12.68	fine sand			
Belvedere Ave.	0.31	5.83	medium sand	0.09	22.52	very fine sand			
Cove Creek Drive	0.92	0.20	coarse sand	0.27	10.75	medium sand			
Manchester Lane	0.22	23.07	fine sand	0.10	21.66	very fine sand			
Monroe Rd.	NS			0.34	10.10	medium sand	4.92	0.37	pebble
Carmel Forest	0.14	8.39	fine sand	0.50	8.98	coarse sand			
Lawton Road RR	0.12	20.98	fine sand	0.15	15.63	fine sand			
Lawton Road	0.38	12.98	medium sand	0.14	20.18	fine sand			
Thermal Road*				0.34	9.49	medium sand	1.15	0.61	very coarse sand
Ranch Road	0.20	9.46	fine sand	0.72	5.52	coarse sand			
Ashely Road 1	0.55	1.50	coarse sand	0.10	9.00	very fine sand			
Ashely Road 2	0.71	0.10	coarse sand	0.50	8.90	coarse sand			
Ashely Road 3	0.59	9.64	coarse sand	0.51	13.89	coarse sand			
Dorn Circle	2.46	0.38	granule	0.27	13.86	medium sand	1.69	0.42	very coarse sand
Orchard Circle	3.53	0.56	granule	0.35	11.94	medium sand			
Eastway Drive	0.50	0.94	coarse sand	0.26	12.14	medium sand	2.62	0.59	granule
Independence Blvd.	0.74	9.37	coarse sand	0.69	5.70	coarse sand			
Linda Lake Drive	3.53	1.82	granule	0.35	11.99	medium sand	2.38	0.83	granule
Average	1.0	7.4		0.3	12.5		2.6	0.6	
Standard Deviation	1.1	7.4		0.2	4.9		1.4	0.2	
CV	120.2	99.9		59.0	39.2		56.6	32.0	

NS = no sample.
*Exposed bedrock no bed sediments.

TABLE 3 Flow Return Intervals (in years)Necessary to Scour Observed Natural Pool Geometry Using FHWA Scour Prediction Methodology and HEC-1 Modeling.

	Depth	Width	Length	Volume
Independence High School	10.62	6.76	2.85	7.04
Manchester Lane	4.30	5.40	1.80	3.80
Monroe Rd.	10.52	9.13	0.64	3.08
Carmel Forest	1.98	1.10	1.63	1.57
Lawton Road RR	2.04	3.17	0.50	1.75
Lawton Road	2.32	2.06	0.65	1.88
Thermal Road	BR*	3.60	1.39	BR*
Ranch Road	4.24	5.72	9.91	4.52
Ashely Road 1	1.46	0.68	0.60	0.97
Ashely Road 3	BR*	0.99	1.34	BR*
Dorn Circle	BR*	1.15	0.59	BR*
Orchard Circle	0.81	1.23	0.90	1.26
Eastway Drive	6.38	6.08	1.18	3.01
Linda Lake Drive	7.94	16.62	7.73	11.18
Independence Blvd.	5.64	1.44	0.32	1.40
Average	4.85	4.34	2.14	3.46
Standard Deviation	3.42	4.28	2.82	2.99
Coefficient of Variation	71	99	132	87

*BR = Bedrock present on pool bottom which would affect maximum scour depth and pool volume.

TABLE 4. Flow Return Intervals (in years) Necessary to Scour Observed Natural Pool Geometry Using USACE Scour Prediction Methodology and HEC-1 Modeling.

	Depth	Width	Length	Volume
Independence High School	2.85	2.52	3.65	2.52
Manchester Lane	0.45	2.24	3.35	1.72
Monroe Rd.	0.53	1.45	1.60	1.15
Carmel Forest	0.67	0.68	1.47	0.93
Lawton Road RR	0.40	0.83	0.87	0.80
Lawton Road	0.43	0.77	1.18	0.86
Thermal Road	BR*	1.88	1.28	BR*
Ranch Road	0.94	2.65	3.32	2.13
Ashely Road 1	0.49	0.46	0.44	0.61
Ashely Road 3	BR*	0.46	0.98	BR*
Dorn Circle	BR*	0.60	0.73	BR*
Orchard Circle	0.26	0.55	0.46	0.55
Eastway Drive	0.82	0.22	1.27	1.26
Linda Lake Drive	0.89	4.10	2.99	3.36
Independence Blvd.	0.23	0.22	0.42	0.37
Average	0.75	1.31	1.60	1.36
Standard Deviation	0.70	1.14	1.14	0.91
Coefficient of Variation	94	87	71	67

*BR = Bedrock present on pool bottom which would affect maximum scour depth and pool volume.

TABLE 5 Comparison of Excavation Costs for Plunge Pools Versus Riprap Culvert Outlet Aprons.

	Pool Excavation	USDA, SCS 1971 Design Criteria	Cost Factor
Independence High School	$137	$95	1.44
Belvedere Ave.	$553	$889	0.62
Manchester Lane	$148	$94	1.58
Monroe Rd.	$411	$1,124	0.37
Carmel Forest	$93	$424	0.22
Lawton Road RR	$1,223	$5,348	0.23
Lawton Road	$877	$4,098	0.21
Thermal Road	$441	$3,827	0.12
Ranch Road	$300	$1,364	0.22
Ashely Road 1	$530	$3,009	0.18
Ashely Road 2	$140	$2,234	0.06
Ashely Road 3	$455	$1,848	0.25
Dorn Circle	$116	$2,309	0.05
Orchard Circle	$608	$2,381	0.26
Eastway Drive	$854	$3,600	0.24
Independence Blvd.	$295	$283	1.04
Linda Lake Drive	$426	$460	0.93
Average			0.47

TABLE 6. Dimensionless Ratios of Plunge Pool Morphology.

	L_{Tw}/Z_{max}	W_{Tw}/Z_{max}	$Vol^{1/3}/Z_{max}$
Independence High School	8.1	5.2	2.4
Belvedere Ave.	10.1	6.4	3.3
Cove Creek Drive	2.7	2.6	
Manchester Lane	9.3	6.5	2.7
Monroe Rd.	7.0	6.2	2.2
Carmel Forest	9.7	3.7	2.3
Lawton Road RR	8.3	7.7	2.9
Lawton Road	9.7	6.2	2.9
Thermal Road	18.9	11.6	4.7
Ranch Road	21.5	7.6	3.3
Ashely Road 1	6.8	3.6	
Ashely Road 2	20.0	13.1	
Ashely Road 3	13.5	6.8	3.0
Average	11.19	6.71	2.96
Standard Deviation	5.66	2.96	0.73
Coefficient of Variation	50.52	44.03	24.64
Maximum	21.47	13.07	4.71
Minimum	2.67	2.62	2.16
Predicted[1]		NT-T[2]	NT-T
Rounded Bed Material	8	6.8-11.5	2.6-3.2
Angular Bed Material	6.9	6-11.5	2.3-3.5

[1]As predicted from flume studies (Opie, 1967).
[2]No tailwater effect (NT); tailwater effect (T).

consistent with the current trend to employ fundamental geomorphologic principles in channel restoration projects.

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