WATERSHED SCIENCE BULLETIN

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The Application of Monitoring and Modeling in Watershed Management

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Integrating Stormwater Controls Designed for Channel Protection, Water Quality, and Inflow/Infiltration Mitigation in Two Pilot Watersheds To Restore a More Natural Flow Regime in Urban Streams

Robert J. Hawley,[a] Matthew S. Wooten,[b] Brandon C. Vatter,[c] Eric Onderak,[d] Mork J. Lachniet,[e] Trent Schade,[f] Geoffrey Grant,[g] Barrett Groh,[h] and John DelVerne[i]*

Abstract

Reducing sanitary sewer overflows (SSOs) is important. But many inflow and infiltration (I/I) mitigation projects simply separate stormwater from the sanitary system and send it downstream without any treatment, causing additional channel erosion in already unstable urban streams. This is unsustainable management of water resources—in terms not only of ecological integrity, but also of public infrastructure, because unstable streams in urban settings impact adjacent sewers and roadways. In a more holistic approach to SSO mitigation, we added goals of water quality and channel protection to two otherwise routine I/I projects. Collecting fluvial geomorphic field data allowed for more accurate estimation of storage volumes required to create a less erosive flow regime in the downstream channel networks. Using continuous simulations over 57 years, we optimized stormwater controls, reducing the total duration of disturbance events and the cumulative sediment transport capacity as close to predevelopment conditions as possible, while meeting the cost criteria of the Sanitation District No. 1 of Northern Kentucky ($0.03/gallon of water treated in a typical year). These collaborative projects demonstrate the benefits of treating I/I mitigation as an opportunity, not only to renew sewer infrastructure in the project area, but also to protect downstream infrastructure from channel erosion, improve water quality by addressing both point and nonpoint source pollution, and benefit aquatic biota by restoring a more natural flow regime. In this setting, stream restoration via flow regime restoration has the potential to be more cost-effective and more beneficial to aquatic biota than approaches that rely exclusively on instream structures, which can be prone to failure in urban and suburban environments.

Introduction

Urban streams face numerous stressors, including altered flow regimes (Poff et al. 2006), physical modifications or burial (Roy et al. 2009), fragmentation (Chin and Gregory 2001), and loss of riparian area or quality (Coles et al. 2010). This degrades the richness and abundance of aquatic resources (Walsh et al. 2005). The mechanisms by which aquatic biota are impacted include chemical (toxicity), physical (habitat), and hydrologic (flow regime) pathways.

The mitigation of chemical stressors from both point sources (e.g., sanitary sewer overflows [SSOs]) and nonpoint sources (e.g., stormwater runoff) is increasing—many communities are investing hundreds of millions to billions of dollars for sewer system upgrades intended to reduce direct overflows of both combined and sanitary sewers as part of enforcement actions (e.g., see US Environmental Protection Agency [USEPA n.d.] for a complete list of enforcement cases). In some communities, these efforts have also included directives to improve the quality of stormwater runoff by, for example, installing best management practices (BMPs) or green infrastructure (GI) in addition to building sewer system capacity. Recognizing the importance of habitat to aquatic communities, some USEPA consent decrees have also included directives to conduct stream restoration projects in addition to more traditional sewer system investments. For this and other reasons, stream restoration expenditures have increased substantially during the last several decades (Bernhardt et al. 2005). But despite large investments in both water quality and habitat improvements, little postconstruction monitoring has occurred, especially in terms of aquatic biota recovery (Bernhardt et al. 2005; Bernhardt and Palmer 2007). Independent lines of evidence suggest that improved water quality and habitat may not be sufficient for preserving/restoring full ecosystem function because many native species depend on features of the natural

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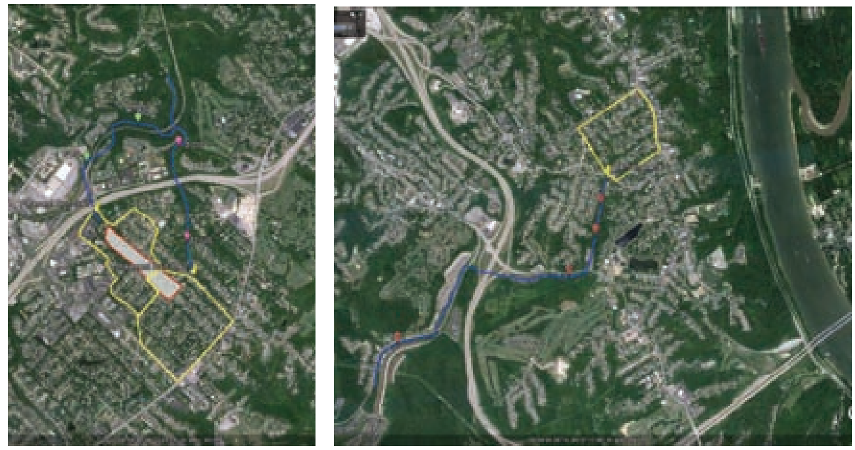
flow regime, such as the frequency and timing of disturbance events (Poff et al. 1997). Thus, a minimum level of hydrologic or watershed restoration might be necessary if functional aquatic communities are a primary goal of such investments (Palmer 2009).

Moreover, the erosive power of the urban flow regime often creates channel instabilities (Bledsoe and Watson 2001; Booth 1990; Hawley et al. forthcoming) that can impact urban infrastructure. In the three Kentucky counties of the greater metropolitan area of Cincinnati, Ohio, channel incision and bank failure have led to the closure and emergency repair of state highways and the complete replacement of main trunk sewers. This sequence, in which poor stormwater management causes channel erosion, which in turn causes damage to urban infrastructure, is highly unsustainable. Recently, the cost of replacing just one exposed sewer crossing on a small stream (~10 feet (ft)[1] [3.0 m] wide) was $100,000. Furthermore, arresting unstable channels with stream restoration that relies heavily on engineered structures, such as cross vanes, is expensive (e.g., $1.25 million for ~600 ft [182.9 m] on a recent project) and can be prone to failure in the urban or suburban setting; dozens of such structures in this area have failed within a few years of construction.



Vernon Lane *Pleasant Run*

Figure 1. Drainage areas (yellow) to project outfalls (push pins), flow paths (blue), and field sites (balloons). I/I project area on Pleasant Run (polygon with white fill, ~32 acres [12.9 ha]) was smaller than project outfall drainage areas (DA1, ~80 acres [32.4 ha]; DA2, ~192 acres [77.7 ha]). I/I project area and drainage area to outfall in Vernon Lane were essentially overlaid (~86 acres [34.8 ha] and ~96 acres [38.8 ha], respectively). North is up. Image courtesy of Google Earth.

In an effort to circumvent this trend, Sanitation District No.1 of Northern Kentucky (SD1) has conducted stream channel stability monitoring, in addition to water chemistry, habitat, hydrologic, and aquatic biota monitoring, as part of its adaptive watershed management strategy when planning and designing system improvements for its combined and separate sewer service areas. In recognition of the interdisciplinary needs of holistic watershed management, this strategy attempts to address multiple sources of pollution that affect water quality, rather than concentrate efforts exclusively on sewer system capacity and overflow reduction.

One common problem in aging sanitary sewer systems is inflow and infiltration (I/I) from nonsanitary sources, such as downspout connections and groundwater infiltration. I/I can become problematic during heavy rains when excess stormwater can overload the sanitary sewer system and cause direct overflows of untreated sanitary waste into receiving streams. Because such untreated waste is considered a human health risk and a water quality pollutant, the Clean Water Act requires that regional sewer agencies ultimately eliminate such SSOs.

This paper describes two recent pilot projects in residential sewersheds with I/I-induced SSOs (Vernon Lane, ~86 acres [34.8 ha], ~29% impervious cover; Pleasant Run, ~32 acres [12.9 ha], ~40% impervious cover; Figure 1) in which SD1 addressed water quality and channel stability design criteria in addition to I/I removal. Water quality goals included a reduction in bacterial loads from both SSOs and stormwater runoff. The channel stability goal was to create a less erosive flow regime in the receiving channels, matching both the peaks and durations of the erosive portion of the predevelopment flow regime to the extent practicable. Our expectation was that a more natural flow regime of high water quality would lead to measurable improvements in downstream aquatic communities.

A central issue in designing stormwater controls for channel protection is the fact that durations of erosive

[1] This paper primarily uses English units because of their dominant use by stormwater professionals in our study area. In some cases, however, industry standards require the use of metric/SI units

flows are typically much longer in the postdevelopment flow regime (e.g., Hawley and Bledsoe 2011), and stormwater controls focused on matching pre development flow durations tend to be more difficult to design than controls focused exclusively on peak flow matching. Even so, Santa Clara, California, requires new developments to match the entire hydrograph, such that postdevelopment flow magnitudes and durations match the predevelopment regime (Santa Clara Valley Urban Runoff Pollution Prevention Program 2004). A similar but simplified strategy in Knox County, Tennessee, uses *centroid-to-centroid* matching of the predevelopment and postdevelopment storm hydrograph for the one-year, 24-hour event (Knox County, Tennessee Department of Engineering and Public Works 2008). This approach could be achieved by controlling and releasing the predevelopment runoff volume for a given storm using primary controls to match the predevelopment hydrograph (i.e., exactly following the blue curve in Figure 2), while storing, infiltrating, and/or evapotranspirating the excess runoff volume using secondary controls. This is desirable for receiving streams because it results in the least hydrologic alteration relative to predevelopment conditions. However, the required footprint of stormwater controls—particularly in areas of poorly drained native soils, such as northern Kentucky—may make the approach difficult to achieve.

A potentially more attainable method for our region currently is erosion control detention (Bledsoe 2002; Figure 2). In this approach, stormwater controls are designed to overcompensate for the excess erosion potential of moderate- and high-frequency storms (i.e., the one- to two-year flows, which are generally considered the flows that most strongly influence channel form [Wolman and Miller 1960]), with the understanding that excess channel erosion may occur during the largest and most infrequent events. We define the flow magnitude where channel erosion begins to occur as the critical flow ($Q_{critical}$). Erosion control detention attempts to match the cumulative erosion potential of the predevelopment flow regime to the extent practicable, without necessarily matching the exact hydrograph of every storm. In other words, the cumulative channel erosion that occurs following development should be similar to the magnitude of channel erosion that would have occurred under predevelopment conditions.

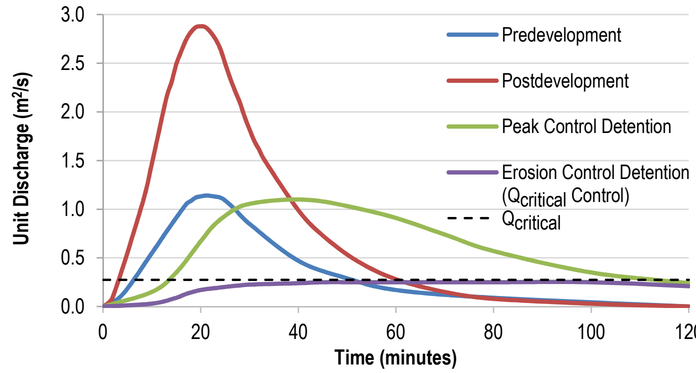


Figure 2. Example of $Q_{critical}$ control erosion control detention in Fort Collins, Colorado, for the two-year, two-hour event (adapted from Bledsoe [2002]), where the two-year storm is overcontrolled such that the cumulative erosion potential of all postdevelopment events more closely matches the predevelopment erosion potential. Peak control detention is defined as detention that is designed to match the predevelopment peak flow magnitude with prolonged duration.

This study explored the potential use of $Q_{critical}$ control as a means to restore more natural flow and disturbance regimes in two receiving streams with channel instabilities largely attributable to upstream urbanization. With limited space in two built-out watersheds, $Q_{critical}$ control focuses on mitigating the erosive portion of the urban flow regime, acknowledging that full hydrologic restoration would probably be cost prohibitive in this case.

Methods

This study used both monitoring and modeling data to evaluate the effectiveness of stormwater controls for reducing downstream erosion impacts in the two pilot project areas, while also improving the biological and water quality condition of the streams. We present a description of geomorphic and biological assessments, along with hydraulic and hydrologic analyses of pre- and postdevelopment flow regimes.

Field Data Collection

This paper evaluated four sites within each project drainage area for biological and geomorphic conditions (Figure 1). Because the I/I project area in Pleasant Run drained to two separate basins (drainage area [DA]1 and DA2 in Figure 1), we divided field sites evenly among the two downstream reaches. We collected preconstruction biological and habitat data according to USEPA rapid bioassessment protocols (Barbour et al. 1999), with regional adaptations by the

Kentucky Division of Water (KDOW 2008). We assessed biological communities using the Kentucky macroinvertebrate biotic index (MBI; Pond et. al 2003).

We conducted fluvial geomorphic field assessments over several stream reaches on project receiving streams to assess channel stability and select suitable sites for data collection. Selected sites were (1) representative of the respective reach and (2) removed from the potential influence of fluvial constrictions, backwater, and channel hardpoints to the extent possible. The latter point was of particular importance because Hawley et al. (forthcoming) documented an increasing risk of channel incision moving upstream from artificial grade control and natural bedrock. In each pilot watershed, we collected cross-section, profile, and bed material data at four sites according to Harrelson et al. (1994) and Bunte and Abt (2001a; 2001b).

Estimating $Q_{critical}$

We estimated $Q_{critical}$ for the median bed material particle size (d_{50}) at each site using the dimensionless shear stress and Manning's equations. We estimated Manning 's *n* using the Cowan method (Chow 1959) and the Shields parameter ($\boldsymbol{\tau}_{*c}$) per Julien (1998). Because both empirical parameters have considerable variability, and limited literature is available on the Shields parameter for embedded clasts of broken limestone bedrock, we populated a range of probable values for both Manning's *n* (e.g., 0.048–0.132) and the Shields parameter (e.g., 0.03–0.54). This produced a range of $Q_{critical}$ estimates, which we summarized by their means and associated 95% confidence intervals.

Estimating Q_2 and Scaling to Project Outfalls

Although we developed detailed hydrologic models of the sewersheds for each project, budgetary constraints did not allow for the extension of those models to the downstream channel locations, except in DA2 of Pleasant Run, where the design site (DA2-upstream [US]) was relatively close to the project outfall. Therefore, for cross-comparison and to enable scaling of $Q_{critical}$ estimates from field sites to project outfalls, we expressed the $Q_{critical}$ estimates as functions of the predevelopment two-year instantaneous peak flow (Q_2) after Watson et al. (1997), using the US Geological Survey (USGS) regional regression equation, which was developed using gage sites with drainage areas as small as ~100 acres (40.5 ha; Hodgkins and Martin 2003):

$$Q_2 = 312 \times DA^{0.673} \qquad \textit{(Eq. 1)}$$

where Q_2 = predevelopment instantaneous peak flow with a recurrence interval of two years, in cubic feet per second (cfs), and *DA* = contributing drainage area in square miles (mi^2).

Sediment Transport Modeling

Hydraulic modeling is a prerequisite to sediment transport modeling because sediment transport equations ultimately depend on hydraulic properties, such as depth, hydraulic radius, and cross-sectional area. Assuming normal depth, we used the Manning's equation to model reach hydraulics, with site-specific hydraulic–geometry relationships after Buhman et al. (2002). We modeled the stream's sediment transport capacity using the Meyer-Peter and Müller (1948) equation as presented by Julien (1998), with corrected parameters from Wong and Parker (2006):

$$q_{bv} = 3.97 \times (\boldsymbol{\tau}_* - \boldsymbol{\tau}_{*c})^{1.6} \times \{(G-1)gd_s^{\,3}\}^{0.5} \qquad \textit{(Eq. 2)}$$

where q_{bv} = unit bedload discharge by volume (m^2/s), which must then be integrated over the top width for the respective flow to determine volumetric bedload (m^3/s); $\boldsymbol{\tau}_*$ = dimensionless shear stress, approximated for gradually varied flow as $\boldsymbol{\tau}_* = RS_f/\{(G-1) \times d_s\}$, where R = hydraulic radius and S_f is approximated by the bed slope; $\boldsymbol{\tau}_{*c}$ = Shields parameter; G = specific gravity of sediment (2.65); g = acceleration of gravity (9.81 m/s^2); and d_s = sediment particle diameter, d_{50} in this application. The equation is presented in SI form for consistency with the referenced presentation in Julien (1998).

Modeling Storm Sewer Hydrology

We developed independent storm sewer models for the Pleasant Run and Vernon Lane project areas using the Storm Water Management Model and Infoworks, respectively, from a combination of field survey, geographic information system data, and connectivity data. We calibrated base models of the existing systems with flow monitoring data, collected over several months, from multiple locations within the respective sewersheds. We then modified these base models to reflect predevelopment and proposed condition scenarios. We took an additional step on the Pleasant Run project to calibrate the predevelopment model to expected peak flows using the rational method. We ran long-term (1950–2007) continuous simulations based on hourly rainfall data from the Covington, Kentucky, airport gage (see National Oceanic and Atmospheric Administration n.d.). Because the time of runoff concentration can be less than one hour on small watersheds, we disaggregated the rain data into five-minute increments for the Pleasant Run model after Ormsbee (1989).

Water Quality Design Parameters

In addition to reducing direct SSOs, a central goal in SD1's watershed plans is to achieve a reduction of at least 50% in nonpoint source bacterial loadings from the first 0.8 inches of stormwater runoff. Moreover, SD1 attempts to achieve these reductions as close to the source as possible. In both pilot watersheds, essentially no stormwater treatment or detention existed in the project areas prior to these projects.

Alternatives Evaluation

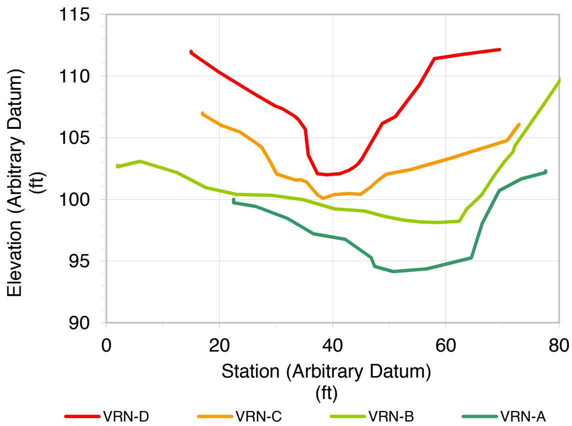
Using the detailed hydrologic models, we developed design alternatives to minimize $Q_{critical}$ exceedances and match the sediment transport capacity of predevelopment conditions to the extent practicable, while also meeting the point and nonpoint source water quality treatment goals. The design alternatives included above ground and below ground multi-stage detention and retention options to reduce erosive flows, coupled with GI to prolong network travel time and reduce nonpoint source bacteria concentrations. GI included down-spout disconnections, curb and walk filter media, curbside or backyard bio-swales and infiltration trenches, pervious pavement, and underground storage in streets. We developed estimates of probable construction cost independently for each project based on regional construction costs.

Results

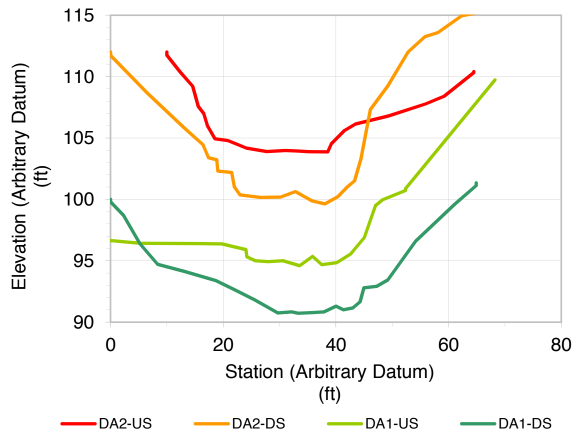
Channel Condition

The receiving streams on both projects had varying degrees of instability. Similar to the findings of Hawley et al. (forthcoming), reaches immediately upstream of hardpoints, such as intact bedrock or exposed pipe crossings, were relatively stable, whereas reaches that lacked the protective capacity of channel hardpoints showed greater instability (Figure 3). This is evident in their cross-sectional forms (Figure 4), where Vernon (VRN)-D, DA2-downstream (DS), and DA1-DS were the farthest removed from hardpoints and tended to have the highest and steepest banks. In contrast, the erosional impacts at VRN-C were minimal because of the protective effects of an exposed pipe crossing (e.g., a hardpoint) at a relatively short distance downstream. The grade-controlling effects of the exposed pipe crossing were also evident at VRN-C by its finer bed material gradation compared to other sites. For example, Figure 5 shows that 50% of the particles were smaller than 30 mm at VRN-C, whereas only ~20% of the particles at the other Vernon sites were

Figure 3. Looking upstream at the DA2-DS site in Pleasant Run (note failure of left bank).

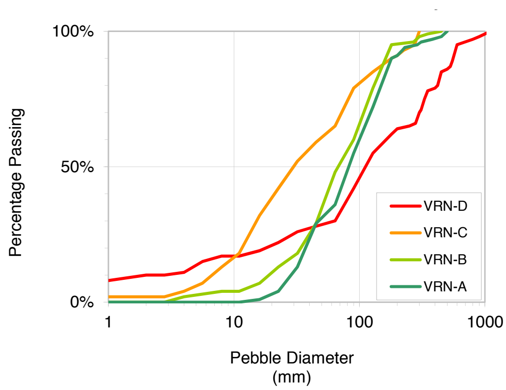


Vernon Lane

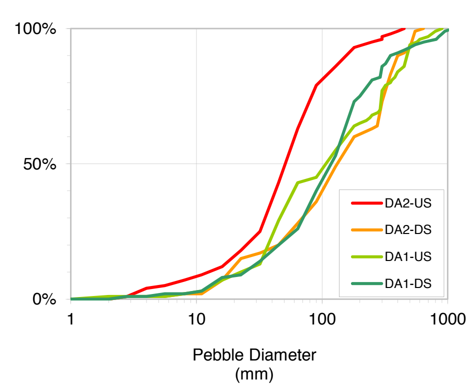


Pleasant Run

Figure 4. Superimposed cross-sections from representative sites (facing downstream, ~3.28 ft = 1 m).



Vernon Lane



Pleasant Run

Figure 5. Superimposed bed material pebble counts from representative sites.

Table 1. Select properties of field sites and mean estimates of $Q_{critical}$.

Site	Drainage Area (mi^2)	Q_2 (cfs)	Slope (%)	d_{50} (mm)	Bankfull Width (ft)	Bankfull Depth (ft)	Critical Depth (ft)	Critical Depth (%BF)	Mean $Q_{critical}$ (cfs)	Mean $Q_{critical}$ (%Q_2)
VRN-D	0.25	122	2.36	113	14	4.17	2.02	48	55.3	54
VRN-C	0.29	136	1.66	30[a]	19	1.96	0.85	43	6.1	5
VRN-B	0.57	214	1.51	68	31	1.86	1.80	97	93.9	50
VRN-A	1.64[b]	435	1.90	83	23	2.63	3.28	125	63.5	14
DA2-US	0.30	139	1.37	52	24	1.73	1.30	75	48.0	35
DA2-DS	0.54	206	3.98[c]	133	22	2.56	1.40	55	43.3	21
DA1-US	0.67	239	1.20	109	19	1.31	4.27[d]	326	793.6	332
DA1-DS	0.79	267	2.71	119	24	2.08	1.61	77	86.0	32

Note: BF, bankfull, ~2.6 km^2 = 1 mi^2; ~35 cfs = 1 m^3/s.
[a] Bed material composition at VRN-C was influenced by a proximate downstream hardpoint (unavoidable in this reach), which induced deposition and caused the bed material to become finer.
[b] VRN-A was less transferable to the project because of the large differences in drainage areas (1.6 mi^2 vs. 0.15 mi^2 project area).
[c] Slope at DA2-DS was possibly over-steepened as a result of active headcutting, despite several attempts to install artificial grade control using cross vanes that were undergoing failure via headcutting and flanking.
[d] Critical depth at site DA1-US was influenced by an atypically wide (40-ft) and flat terrace accessed at a depth of only 1.3 ft.

smaller than 30 mm; this indicates that the flatter bed slope upstream of the pipe crossing had induced sediment deposition at VRN-C. Table 1 summarizes select metrics.

Preconstruction Habitat and Biological Conditions

Biological conditions of each stream, based on habitat assessments (Table 2) and macroinvertebrate communities (Table 3), indicated generally degraded conditions in receiving streams of both project areas. Habitat was designated, after KDOW (2008), as "nonsupporting" of aquatic life at all sites. Macroinvertebrate communities, again after KDOW (2008), were designated as "poor" at VRN-A, VRN-B, and VRN-D, and VRN-C was "very poor;" in the Pleasant Run project area, site DA2-DS was rated as "poor," and all three remaining sites were "very poor."

Table 2. Habitat assessment scores.

Site	ES	EMB	VDR	SD	CFS	CA	FOR	BS		VP		RZW		Score	Classification
								Left	Right	Left	Right	Left	Right		
VRN-D	11	7	12	9	12	9	15	5	3	5	3	7	2	100	Nonsupport
VRN-C	9	9	9	6	8	8	16	7	5	6	3	9	2	97	Nonsupport
VRN-B	13	13	10	7	10	11	13	6	4	6	3	6	1	103	Nonsupport
VRN-A	17	12	12	11	15	13	7	4	3	4	3	1	1	103	Nonsupport
DA2-US	10	10	9	8	12	11	14	6	7	6	6	4	3	106	Nonsupport
DA2-DS	10	16	11	5	9	13	16	5	3	9	5	8	2	112	Nonsupport
DA1-US	10	8	8	3	7	12	13	7	2	4	4	8	2	88	Nonsupport
DA1-DS	10	7	9	7	10	15	15	7	5	7	5	8	5	110	Nonsupport

Notes: ES, epifaunal substrate; EMB, embeddedness; VDR, velocity/depth regime; SD, sediment deposition; CFS, channel flow status; CA, channel alteration; FOR, frequency of riffles; BS, bank stability; VP, vegetative protection; RZW, riparian zone width.

Table 3. Kentucky macroinvertebrate metric and index scores.

Site	G-TR	G-EPT	mHBI	%Ephem*	m%EPT	%C+O	%CLING[a]	MBI	Classification
VRN-D	6	0	7.48	N/A	0	12.5	0.24	18.69	Poor
VRN-C	10	0	7.88	N/A	0	22.5	0.5	17.54	Very Poor
VRN-B	18	2	7.58	N/A	0.5	16.9	3.2	22.34	Poor
VRN-A	15	2	7.71	1	1	8	2.5	22.54	Poor
DA2-US	14	0	7.97	N/A	0	82	3	10.25	Very Poor
DA2-DS	11	2	7.82	N/A	1.2	23.3	0.6	18.91	Poor
DA1-US	9	1	7.08	N/A	0	94.7	2.6	9.23	Very Poor
DA1-DS	15	1	6.38	N/A	4.81	88.7	6.5	14.35	Very Poor

Notes: G-TR, genus-level taxa richness; G-EPT, genus-level Ephemeroptera, Plecoptera, and Trichoptera taxa richness; mHBI, modified Hilsenhoff biotic index; %Ephem, relative abundance of mayflies, only used in headwater stream assessments; m%EPT, relative abundance of EPT individuals, minus the genus Cheumatopsyche; %C+O, relative abundance of Chironomidae and Oligochaeta; %CLING, relative abundance of clingers.*
[a] Note the particularly low abundance of clingers, a habitat type that is indicative of the relative stability of the channel.

Estimates of $Q_{critical}$

Based on a range of probable estimates for the empirical parameters of Manning's *n* and the Shields parameter, we produced a range of $Q_{critical}$ estimates with the mean values shown in Table 1. Because each site had different contributing drainage areas, we expressed each $Q_{critical}$ estimate as a percentage of Q_2 for greater comparability among estimates (see Table 1, far right column). As discussed above, VRN-A, VRN-C, DA1-US, and DA2-DS were all influenced by factors that could artificially bias the $Q_{critical}$ estimate (see Table 1, notes). As such, VRN-D and VRN-B were most representative for design on the Vernon Lane project, with mean estimates of ~50% of Q_2. That is, the $Q_{critical}$ values corresponded to approximately half of the predevelopment, two-year peak flow magnitude (Q_2). In Pleasant Run, sites DA1-DS and DA2-US were most representative, with mean estimates of ~30% of Q_2.

We then scaled the respective $Q_{critical}$ values of ~40 cfs (1.13 m^3/s) at VRN-D, ~66 cfs (1.87 m^3/s) at DA1-DS, and ~37 cfs (1.05 m^3/s) at DA2-US upstream to the respective project outfalls to develop project design values using the USGS regional equation for Q_2 (Hodgkins and Martin 2003) after Watson et al. (1997; Table 4).

Table 4. Design $Q_{critical}$ values scaled to project outfalls via $(DA_{project}/DA_{stream})^{0.67}$.

Stream Site	Stream Drainage Area (mi²)	Stream Design $Q_{critical}$ (cfs)	Project Drainage Area (mi²)	Project Design $Q_{critical}$ (cfs)
VRN-D	0.25	40	0.15	28
DA1-DS	0.79	66	0.13	20
DA2-US	0.30	37	0.30[a]	37

Note: 1 mi² ≈ 2.6 km²; 1 m³/s ≈ 35 cfs.
[a] Because of the close proximity of DA2-US to the Pleasant Run project outfall, the detailed hydrologic model was extended downstream to encompass the entire drainage area of DA2-US, requiring no flow scaling in this case.

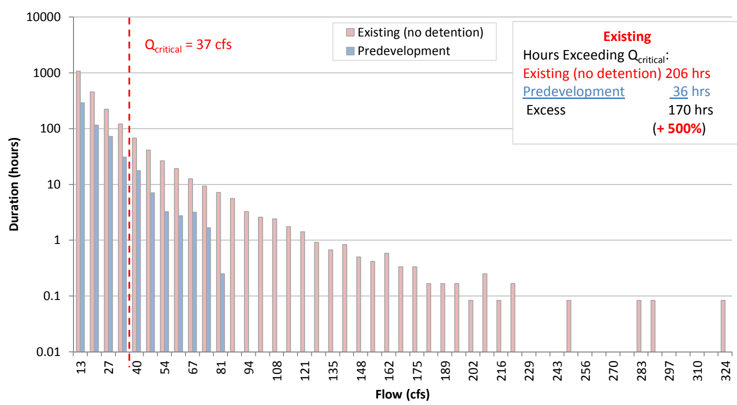


Figure 6. Magnitude and duration of $Q_{critical}$ exceedances under existing and predevelopment conditions in DA2 of Pleasant Run over 57 years of rainfall, ~35 cfs = 1 m³/s.

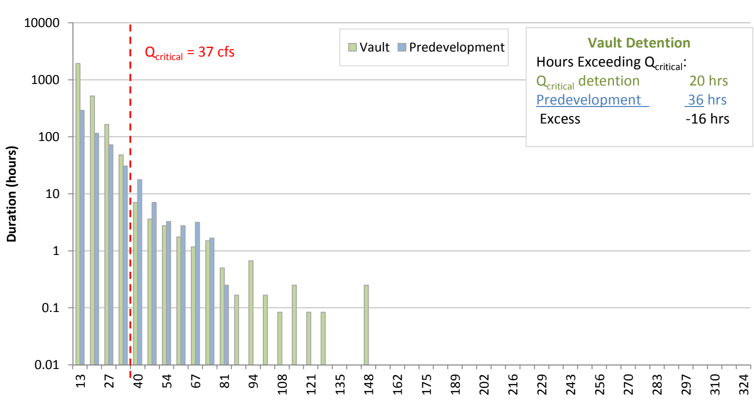


Figure 7. Magnitude and duration of $Q_{critical}$ exceedances under vault detention and predevelopment conditions in DA2 of Pleasant Run over 57 years of rainfall, ~35 cfs = 1 m³/s.

Hydrologic Simulations

We modeled 57-year simulations of predevelopment, existing (postdevelopment with no flow control), and several proposed stormwater control scenarios to determine their performance in minimizing cumulative $Q_{critical}$ exceedances (Table 5). Despite differences in modeling platforms and rainfall resolution, both projects showed substantial imbalances between existing and predevelopment conditions. DA2 of Pleasant Run (~40% imperviousness) had 206 hours of $Q_{critical}$ exceedances under existing conditions compared to 36 hours under predevelopment conditions, for an excess of 170 hours, or 500% (Figure 6). In DA1 of Pleasant Run (~40% imperviousness), the values for existing and predevelopment conditions were 275 hours and 25 hours, respectively, for an excess of 250 hours. In Vernon Lane (~29% imperviousness), the values were 95 hours compared to 0 hours, for an excess of 95 hours.

Given the magnitude of the existing hydrologic alteration, it seemed impractical, in some cases, to control stormwater to predevelopment conditions (i.e., by installing controls such that, above 37 cfs (1.05 m³/s), the red bars would match the blue bars in Figure 6). However, the purpose of this exercise was to see what level of control (and associated costs) would be required to achieve a more natural flow regime. Because of the heavily urban nature of the project areas, large footprints were not readily available to fit more cost-effective detention structures. For example, in DA2 of Pleasant Run, a traditional detention basin augmented with subsurface vaults was required to nearly match predevelopment flow conditions (Figure 7; note that the green bars come much closer to matching the blue bars above 37 cfs). But perhaps an equally valuable consideration when assessing the performance of various design scenarios is the improvement relative to existing conditions, especially given that these channels have been adjusting to altered flow regimes for more than 50 years. For example, even the smallest detention alternative in DA2 of Pleasant Run (i.e., graded detention in Table 5) reduces the duration of $Q_{critical}$ exceedances by more than 60% (or 75 hours) relative to existing conditions (206 hours).

Table 5. $Q_{critical}$ exceedances, cumulative sediment transport capacity, and estimated costs of competing design scenarios for DA2 in Pleasant Run, modeled over 57 years of rainfall.

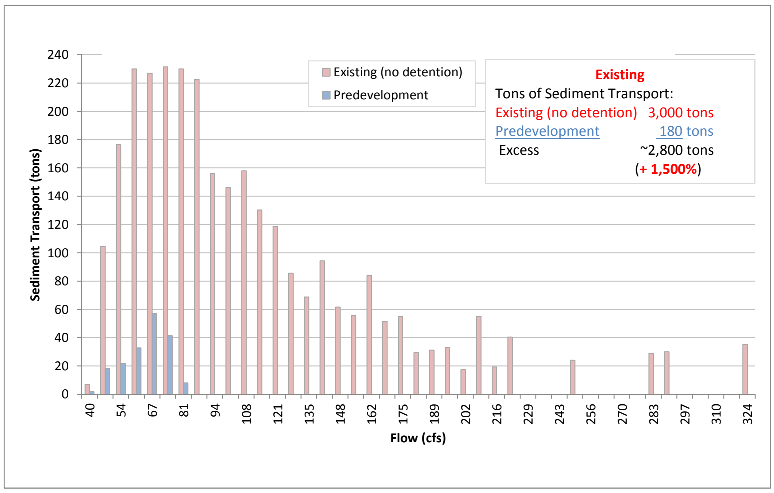
Model Scenario[a]		Storage Volume	Cost		$Q_{critical}$ Exceedance		Sediment Transport	
Name	Description	(Thousands of ft³)	Total Cost ($k)	Mean Annual Cost per Gallon Stored[b] ($/gal)	Duration (hours)	Relative to Predvlp.	Total (tons)	Relative to Predvlp.
Predevelopment	Predevelopment conditions		—	—	36	—	180	—
Existing	Existing conditions (no detention)		—	—	206	+500%	3,000	+1,500%
Graded Detention	Detention basin with graded side slopes	49	140	0.002	75	+100%	1,400	+650%
Graded Detention with Inline Basin	Graded basin with downstream inline basin	79	170	0.002	53	+50%	930	+400%
Wall and Graded Detention	Graded basin augmented with retaining wall	95	200	0.003	40	+10%	660	+265%
Wall Detention with Inline Basin	Retaining wall basin with downstream inline basin	125	230	0.003	30	−15%	450	+150%
Vault Detention	Detention basin with subsurface vaults	292	2,000	0.030	20	−45%	240	+33%

Note: ~35 ft³ = 1 m³; ~0.264 gallons (gal) = 1 L; ~1.1 ton = 1 metric ton.
[a] Implicit in each design are water quality features (e.g., bio-infiltration) to achieve the water quality criteria for nonpoint source pollution of removing 50% of bacterial loads from runoff induced by the first 0.8 inches (~2 cm) of precipitation.
[b] Mean annual cost per gal stored during a typical year of precipitation (i.e., 1970 rainfall record).

Cumulative Sediment Transport

Evaluating design alternatives based exclusively on durations of $Q_{critical}$ exceedances can mask potentially disproportionate increases in erosive power at the highest flow events. For example, the 15 minutes of flows at 148 cfs (4.19 m^3/s) in the vault design (Figure 7) could do nearly four times the damage of 15 minutes of flows at 81 cfs (2.3 m^3/s) under the predevelopment scenario. Indeed, sediment transport modeling showed that the flows at 148 cfs (4.2 m^3/s) could transport 31 tons (28.1 metric tons) of sediment, whereas the same 15 minutes at 81 cfs (2.3 m^3/s) could transport only about 8 tons (7.3 metric tons) of sediment. Designing controls to match the cumulative sediment transport capacity of predevelopment conditions may be more appropriate than matching only the duration of $Q_{critical}$ exceedances because it may be a better surrogate for channel stability, and would more effectively match the natural habitat disturbance regime of the predevelopment setting.

For example, when integrating over the 57-year simulation, Table 5 indicates that the wall and graded detention alternative in DA2 comes within 10% of matching the total number of hours of $Q_{critical}$ exceedances in predevelopment conditions. However, it still has the potential to transport 265% more sediment than in predevelopment conditions. Although the design is a vast improvement over existing conditions (in which sediment transport capacity is 1,500% more than in predevelopment conditions; Figure 8a), it exemplifies the importance of considering cumulative sediment transport in addition to $Q_{critical}$ exceedances. (See Figure 8b for the vault detention alternative.)



(a) Existing and Predevelopment

(b) Vault Detention and Predevelopment

Figure 8. Cumulative sediment transport capacity in DA2 of Pleasant Run over 57 years of rainfall.

Cost–Benefit Analysis

Based on previous evaluations of alternative approaches for meeting its water quality goal for nonpoint bacterial pollution (50% reduction from the first 0.8 inches [~ 2 cm] of precipitation) in its separate sewer service area, SD1 has a watershed planning goal of keeping the capital costs of stormwater controls associated with both peak flow reduction and water quality improvement below \$0.03/gallon of runoff[2] treated per typical year (compared to \$0.50/gallon in the combined sewer service area). We had limited cost criteria data from other communities; however, our water quality alternatives evaluation identified this target of \$0.03/gallon treated as the *knee of the curve*, in that unit costs of associated BMPs increased at much faster rates above the \$0.03/gallon value, whereas BMPs below the \$0.03/gallon value tended to have similar cost-effectiveness in the separate sewer service area. We estimated the cost-effectiveness by running a continuous simulation of the typical-year rainfall (i.e., 1970), and determining how many total gallons would be effectively routed through stormwater controls. All design scenarios on all projects achieved the \$0.03/gallon criterion; however, the projects had considerable variability because of site constraints. For example, a graded basin augmented with a retaining wall could effectively match

[2]~0.264 gallons = 1 liter.

predevelopment $Q_{critical}$ exceedances for $200,000 ($0.003/gallon), but it would take a $2 million ($0.030/gallon) basin with subsurface vaults to come within 33% of matching the predevelopment sediment transport capacity (Figure 9).

Discussion

The project areas were developed primarily in the 1950s and 1960s with no stormwater detention. This led to large increases in the magnitudes and durations of erosive flows and much higher sediment transport capacity, causing severe instabilities in receiving stream reaches that lack the protective capacity of grade control. System-wide instability was so severe that several reaches with recently installed cross vane grade-control structures were already being undermined by headcutting and/or flanking at the start of this project.

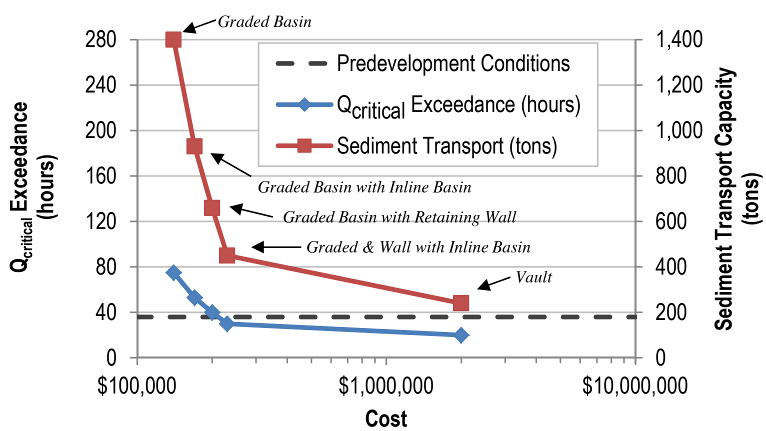


Figure 9. Performance vs. cost of detention alternatives in DA2 of Pleasant Run.

As a part of its I/I mitigation projects, SD1 looked for opportunities to install stormwater controls that could help arrest the downstream channel instability by restoring a less erosive flow regime. The storage requirements for detention that could result in a predevelopment-like sediment transport regime were relatively large (e.g., ca. 300,000 ft^3 [8,495 m^3] in DA2 of Pleasant Run), and SD1 found very few opportunities to retroactively fit controls of such scale. We considered an array of distributed and centralized controls, such as pervious pavement, swales, and underground storage, but multistage detention was typically the only control that could store the required volume at SD1's cost criterion of $0.03/gallon. DA2 of Pleasant Run and Vernon Lane included just enough open space for surface detention that could be optimized for $Q_{critical}$ control and augmented with bio-infiltration to meet our water quality design criteria for nonpoint source pollution (removal of 50% of bacterial loads from runoff induced by the first 0.8 inches of precipitation).

Because DA1 of Pleasant Run included no open space, the only locations that could hold the required volume of ca. 250,000 ft^3 (7,079.2 m^3) were in open-channel sections. We were uncertain how the potential benefits for downstream water quality, habitat, and channel stability would be received by the permitting authorities at the US Army Corps of Engineers and KDOW, given their general resistance to inline storage basins. A request for the consideration of inline storage seemed to be warranted in this case because of the heavily degraded and intermittent state of these few hundred feet of channels that were not otherwise buried during the original construction in the 1950s relative to the potential system-wide benefits. If this aspect of the project is not permitted, our sites downstream of DA1 will serve as controls relative to DA2 and Vernon Lane, where designs are less dependent on permitting considerations.

Beyond the $0.03/gallon criterion, we also considered the relative cost-effectiveness of the various basin designs. For example, adding subsurface vaults in DA2 would bring the sediment transport regime to within 33% of predevelopment conditions, but the costs were an order of magnitude higher than the next best alternative that controlled to within 150% of the predevelopment regime. Given that the existing conditions were 1,500% more erosive than the predevelopment regime, the knee of the cost curve ($220,000) in Figure 9 seemed to be a reasonable selection.

Conclusions

Numerous studies have demonstrated that watershed urbanization directly alters the quality, habitat, and stability of receiving streams, a finding further supported by our study. However, by attempting to mitigate these impacts as a part of I/I mitigation projects, our approach may be novel. Sanitary sewer systems and stormwater quantity and quality have traditionally been approached as separate design problems requiring different engineering teams. But we did more than simply consider design criteria from all three fields—our stormwater controls are actually calibrated to their respective receiving streams. By collecting fluvial geomorphic data, we were able to more accurately estimate how much volume we needed to control to promote downstream channel stability. And rather than engineering the stream channel with expensive grade-control structures, we are promoting the more holistic restoration of the fluvial geomorphic process by designing to a flow regime that better matches the natural disturbance regime and is of high water quality.

In future work, we expect to quantify improvements in channel stability and macroinvertebrate communities with our planned postconstruction monitoring and will revisit the metrics summarized in Tables 2 and 3.

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