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Low Impact Stormwater Treatment Methods

APPLICATION OF A SIMPLIFIED ANALYSIS METHOD FOR NATURAL DISPERSION OF HIGHWAY STORMWATER RUNOFF

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16. ABSTRACT

This paper focuses on evaluating natural dispersion runoff infiltration performance by utilizing simulated rainfall/runoff data collected using a field-scale rainfall simulator coupled with a numerical model to study the effects of slope length, angle, and impervious contributory area on natural dispersion applications. A simplified equation was established, termed the LID Design Equation, to analyze natural dispersion performance based on multiple variables that can be determined for site specific conditions, allowing highway engineers to tailor natural dispersion requirements to various locations throughout Washington. Furthermore, the research and resulting evaluation procedure indicate that current evaluation procedures for the use of natural dispersion as a viable stormwater quantity control strategy are not physically accurate.

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EXECUTIVE SUMMARY

Stormwater quantity control has long been a challenge for highway designers. Traditionally centralized best management practice (BMP) designs are often cost prohibitive and inefficient in many rural highway applications. The use of existing vegetated rights of way as a method of treating stormwater, a component of the broader stormwater treatment concept more generally referred to as "low impact development" (LID), has become a primary focus of the Washington State Department of Transportation. In order to adequately design and utilize such stormwater management controls, however, further research and correlation between numerical infiltration/runoff models and field experiments were needed. This paper focuses on evaluating natural dispersion runoff infiltration performance by utilizing simulated rainfall/runoff data collected using a field-scale rainfall simulator coupled with a numerical model to study the effects of slope length, angle, and impervious contributory area on natural dispersion applications. A simplified equation was established, termed the LID Design Equation, to analyze natural dispersion performance based on multiple variables that can be determined for site specific conditions, allowing highway engineers to tailor natural dispersion requirements to various locations throughout Washington. Furthermore, the research and resulting evaluation procedure indicate that current evaluation procedures for the use of natural dispersion as a viable stormwater quantity control strategy are not physically accurate.

Specifically, it was determined that:

- Over the range of slopes and system specific conditions studied, runoff did not correlate positively to slope (i.e., higer runoff was not observed at higher slopes). Most importantly, the current design guidline requiring LID slopes to be not steeper than 7:1 is not justified. Application of this guidline eliminates many areas that have been shown to yield

accepatble levels of LID infiltration performance or increases the LID length well beyond what is necessary at many roadside locations.

- Saturated conductivity (Ks) was found to be the most critical design parameter. Required minimum values of Ks (102 mm/hr) found in existing natural dispersion guidance were found to be not physically based, again eliminating areas that are well suited for LID application or resulting in LID lengths greater than needed in some situations.

As a result of this research, a simplied, user friendly design equation (LID Design Equation) was developed and shown to be an accurate means of determining LID length by comparison to field data and calibrated model simulations using a finite difference solution to the hydrodynamic wave equaiton dynamically coupled with the Green-Ampt infiltration equation.

INTRODUCTION AND BACKGROUND

Development and urbanization is a primary cause of many adverse environmental impacts in the United States and abroad. One critical aspect of these impacts is related to stormwater runoff from man-made impervious areas, such as buildings, subdivisions, parking lots, and roadways (Booth and Jackson 1997; Lee and Heaney 2003; USEPA 2000) . Many research efforts have been undertaken in recent years to determine the characteristics of stormwater impacts on the natural environment; it has been generally concluded that stormwater from pollution generating impervious surfaces (PGIS) must be mitigated to ensure that the quantity and quality of that runoff will not further degrade baseline environmental conditions (Booth et al. 2002; Lee and Heaney 2003; Massman 2003; Tilley et al. 2003; USEPA 2000; WSDOE 2001; WSDOE 2004).

Traditional methods of stormwater runoff quantity treatment from PGIS have focused on centralized detention-based best management practices (BMPs) that reduce the amount of stormwater released from a developed site (Prince George's County 1999). Stormwater quality treatment – which targets pollutant removal from the runoff water – has also utilized systems that focus primarily on detention and precipitation of pollutants by settling of suspended solids, sometimes coupled with biological processes at centralized locations (Massman 2003). These traditional methods, while generally effective in achieving established treatment goals, have various drawbacks – particularly maintenance and property costs – which encourage environmental engineers and scientists to search for alternative methods of stormwater mitigation. Studies have shown that detention based BMPs inhibit the natural recharge of groundwater to localized areas, may not meet new stormwater regulations imposed by state and federal agencies, and are very susceptible to performance problems when maintenance practices are neglected (Massman 2003; Prince George's County 1999).

A relatively recent concept in stormwater management is emerging as a result of increased interest placed on stormwater impacts by regulatory agencies and the public. Known as "low impact development" (LID), this new technology first pioneered by Price George's County in Maryland utilizes natural drainage and infiltration systems coupled with engineered site designs to treat stormwater runoff quantity and quality at or near the PGIS that contributes runoff (Prince George's County 1999). By using decentralized treatment regimes such as rain gardens, grass filter strips, or natural bioretention cells located adjacent to runoff producing PGIS, the focus of LID is to reduce conveyance costs (i.e. pipe, catch basins), provide low-maintenance treatment areas, and improve the aesthetic aspect of stormwater management in urban areas. Although originally envisioned for use in urban development, LID principles show promise for incorporation into transportation facilities as an economical and environmentally sound stormwater management practice (Tilley et al. 2003; USEPA 2000).

Methods of stormwater mitigation are comprised of two basic elements: stormwater quality treatment (to improve the runoff water quality by reducing contaminant concentrations) and stormwater quantity treatment (to reduce the total volume of stormwater resulting from conversion of permeable soils to impermeable pavements). Numerous studies and research efforts have been performed in recent years to determine the performance and design characteristics of many of the LID strategies currently being used for highway stormwater treatment (Backstrom 2003; Barber et al. 2003; Barrett et al. 1998b; Barrett et al. 1998a; CALTRANS 2003; Cristina and Sansalone 2003; Deletic 2001h; Deletic 2004; Dierkes and Geiger 1999; Kaighn and Yu 1996; Mikkelsen et al. 1997; Munoz-Carpena et al. 1999a; Prince George's County 1999; Sansalone 2004; Tilley et al. 2003; United States EPA 2000; USEPA 2000; Yonge 2000; Yonge and Newberry 1996). A vast majority of work found in the literature

is focused on quality treatment of highway runoff; most generally regarding contaminant and sediment retention capabilities of roadside grassed swales and filter strips designed for runoff quality treatment. However, in many roadway settings the attenuation of the total volume of highway stormwater runoff is of particular interest, especially in rural areas where the extensive linearity and length associated with the highway system preclude the efficient and practical use of more traditional centralized detention-based quantity control facilities (i.e. detention and infiltration ponds and constructed wetlands). Centralized BMPs such as these are problematic in the rural highway system as containment and conveyance of runoff must first be achieved in order to transport the stormwater to the quantity and/or quality treatment facility. These systems, with their numerous components necessary to ensure effectiveness (curb and gutters, catch basins, drain pipes, detention ponds, and overflow outfalls) have relatively high continuous maintenance needs and have been subject to numerous failure issues due to poorly estimated design infiltration rates and hydraulic conveyance issues (Massman 2003). Detention-based quantity control BMPs have also been shown to be marginal in their ability to protect downstream receiving waters and also ineffective in groundwater recharge (Booth et al. 2002; Booth and Jackson 1997; Massman 2003). The use of LID BMP design in lieu of traditional stormwater treatment is beneficial not only in its more aesthetic acceptance by the public but also in its ability to reduce maintenance needs and increase groundwater recharge (Prince George's County 1999; USEPA 2000).

Current LID Design Guidance for Highway Stormwater Mitigation

General Quality/Quality Control

Current design guidance available for highway design use in Washington State can be found in several documents, including the Washington State Department of Ecology's (WSDOE) *Stormwater Management Manual for Western Washington, Stormwater Management Manual for*

Eastern Washington, and the WSDOT's *Highway Runoff Manual*. Numerous design parameters have been established in these and other guidance sources for the use of LID principles such as grassed filter strips, grassed swales, infiltration/exfiltration trenches, and amended soils for the treatment of stormwater quality (contaminant retention) adjacent to the roadway edge (WSDOE 2001; WSDOE 2004; WSDOT 2004b). As mentioned previously, the amount of research specific to quality treatment using LID methods is extensive and the basis for design criteria associated with these BMPs is understandable. However, the use of these types of BMPs for the total containment of highway runoff (quantity control) is not well documented in the literature; as a result, the design guidance as currently established in Washington State's stormwater manuals is not fundamentally based, but rather engineering "judgment" and existing runoff quality treatment guidelines have been used for guideline development (Personal Communication – Steve Foley, King County Water and Land Resources Division, Seattle WA., October 2003; Personal Communication – Karen Dinicola, WSDOE Municipal Stormwater Unit, Olympia WA., October 2003; Unpublished Meeting Notes – WSDOT, LID Credit Committee Meeting, Olympia WA., October 2004). As a result, the use of existing quantity control LID design guidance may be suspect in its applicability and effectiveness.

Natural Dispersion

As a quantity control LID measure, "natural dispersion" is the process of treating stormwater by infiltration in roadside areas immediately adjacent to the roadway edge. Dispersive measures include the interception of sheet flow runoff along the length of the roadway without containment or conveyance and allowing for the hydrologic capacity of the roadside soils to effectively infiltrate the stormwater (WSDOT 2004b). Current natural dispersion design guidance provided by WSDOE and WSDOT is summarized in Table 1.

WSDOT is encouraged to incorporate LID methods to treat stormwater runoff volumes on rural linear transportation facilities; however, several of the natural dispersion design parameters included in the guidance sources eliminate the use of this method at most highway locations in Washington State. The specific requirements for slope angles and saturated hydraulic conductivity (K_s, L/T) for the adjacent roadside treatment areas (hereafter termed "LID areas") may be too conservative and are not easily achievable within highway rights-of-way. While the benefits (both economic and hydrologic) of using natural dispersion on rural transportation facilities are apparent, the applicable design methods used to determine their suitability are questionable.

Of particular interest to highway designers is the slope requirement necessary to achieve effective natural dispersion. As stated in the WSDOT *Highway Runoff Manual*, side slopes for LID treatment areas can not exceed a 7:1 horizontal/vertical ratio (WSDOT 2004b), although slopes preceding LID areas (but not included in the overall LID length) can by 4:1. Unfortunately, the 7:1 slope requirement eliminates the incorporation of roadway embankments into the total LID area available within the existing rights of way (WSDOT 2004a). Design slopes for roadway embankments are between 6:1 and 2:1 for most applications, with an overwhelming majority of slopes flatter than 4:1 being located on major divided highways and interstates (WSDOT 2004a). Rationale behind limiting the LID treatment area slope to 7:1 or less is based primarily on two factors: slopes included in previous research efforts for the use of grassy swales for roadway runoff quality control rarely exceeded 10:1 (Backstrom 2003; Barrett et al. 1998d; Deletic 2001g; Deletic 2004; Munoz-Carpena et al. 1999b; Yonge 2000), and concern over erosion and flow concentration on slopes greater than those already deemed acceptable for use as quality control facilities (WSDOT 2004b). As mentioned previously

however, according to the WSDOT *Highway Runoff Manual*, roadway side-slopes upstream from an acceptable LID area can be as steep as 4:1 (WSDOT 2004b). This fact brings into question the concern over flow concentration on steeper slopes as being a viable issue.

Table 1. Current Natural Dispersion Guidelines

	WSDOE - Stormwater Management Manual for Eastern Washington (2004)	WSDOE - Stormwater Management Manual for Western Washington (2001)	WSDOT - Highway Runoff Manual (2004)
Impervious Area Slope:	<15%	<15%	<15%
LID Area Slope:	12.5:1 (8%) or increase LID length by 1.5 ft per 1% increase in slope above 8%	12.5:1 (8%) or increase LID length by 1.5 ft per 1% increase in slope above 8%	7:1 (14%) max.
Length Ratio (impervious/LID area):	10 ft for first 20 ft of impervious; 5 ft for each additional 20 ft or fraction thereof	10 ft for first 20 ft of impervious; 5 ft for each additional 20 ft or fraction thereof	10 ft for first 20 ft of impervious; ratio of 4:1 for each additional foot of impervious thereafter
Infiltration Rate (in/hr):	N/A	N/A	4 in/hr min.

Current Research Goals

For reasons stated above, the Washington State Department of Transportation has been very interested in determining the applicability and performance of LID methods for use in rural areas. The WSDOT manages over 18,000 lane miles of highway in Washington State, a majority of which are located in rural areas (CH2M-Hill Staff 2001; WSDOT Transportation Data Homepage 2004). As a budgetary item, the agency's biennial investment in stormwater treatment related to design and maintenance approaches $10M, with no good estimate on actual construction and right-of-way costs associated with these activities (CH2M-Hill 2001). In an effort to control these costs and investigate methods of improving stormwater treatment efficiency, the WSDOT maintains an ongoing research and development program to pursue new

and innovative ways to handle stormwater generated on its facilities. Of key importance in these efforts is the determination of stormwater quantity treatment effectiveness of existing roadside embankments, specifically the use of natural dispersion. The use of typical roadway embankments is presently not allowed by the current design guidance.

Without regulatory guidance allowing for the inclusion of roadway embankments as a practical runoff quantity control strategy, hydrologic modeling of such embankments would be necessary to demonstrate their effectiveness or their inclusion into the natural dispersion design criteria. According to WSDOT however, there is a lack of modeling methods currently available to quantify stormwater losses along roadside areas due to natural dispersive and infiltrative mechanisms; a situation which potentially leads to the design of excessively large detention facilities if they are required (Yonge 2003). The effects of scale are equally important to hydrologic analysis as previous research has shown (Joel et al. 2002a). Use of traditional runoff modeling equations such as The Rational Method can result in significant error when analyzing smaller catchments similar to those found along roadside areas. A better methodology based on actual field data and modeling is needed.

The purpose of this research project was to investigate the relationship between roadway embankment slope (length and angle), soil properties (soil texture, vegetative cover, and hydraulic conductivity), rainfall intensities, and contributory impervious area (roadway width) to infiltration and runoff. Coupled with field data, a numerical model capable of analyzing dispersed stormwater runoff from roadway pavements (and the infiltration of that runoff into roadway embankments) was also incorporated into the analysis; this allowed for extrapolation of empirical data to various roadway and environmental situations. The final results of the research show that existing guidelines for the incorporation of natural dispersion are inaccurate for many

roadway configurations in rural areas of Washington State and also, based on the data collected during this study, the concern over negative effects of increased slope on the capabilities of the LID area to attenuate highway runoff are unwarranted.

EXPERIMENTAL METHODS

The investigational approach of this study included two primary efforts: (1) field experiment data collection and (2) numerical model calibration and validation. By using two separate methods it was anticipated that the computer simulation could be calibrated and verified by the field data, allowing for further investigation of the effects of the various parameters (rainfall intensity, roadway width, LID area length, soil properties, and slope angle) beyond the constraints of the field experiments using the computer model. Determination of a simplified relationship between the design parameters, capable of being applied by highway design professionals without the need for extensive computer modeling, was also a primary consideration of the chosen research methodology.

Field Experiments

The first task to be completed was the data collection component, which consisted of two strategies: obtaining real-world rainfall/runoff data at select highway locations (static site monitoring) and generating synthetic data using a rainfall/runoff simulation method at various other locations (simulation). At all locations, "natural" roadway embankments (i.e. those embankments not specifically designed for stormwater treatment such as grassed swales or filter strips) were analyzed to determine their runoff treatment capabilities. By using both actual rainfall/runoff data and simulated data, the relationships between actual natural dispersion performance and that observed in simulation could be determined. Although the primary objective of the field experiments was to verify and calibrate the computer model (described in a

subsequent section titled *Numerical Modeling*), it could be argued that relying completely on field simulation data may not represent the realities of the natural events.

Static Site Monitoring

Static site monitoring was accomplished by the selection of three separate highway locations in Washington State with differing soil and climactic characteristics. Two sites were located in eastern Washington; Site 1 was located on US-2 north of Spokane (MP 298.4), Site 2 was located on SR-270 east of Pullman (MP 6.3). The third site (Site 3) was located in western Washington on I-90 west of Snoqualmie Pass (MP 45.7). Each site differed in annual precipitation, with average annual tabulated values of 18in, 24in, and 90in (460cm, 610mm, and 2290mm), respectively (WSDOE 2001; WSDOE 2004). Contributory impervious area and site layout varied at each location as shown in Figure 1 and reported in Table 2.

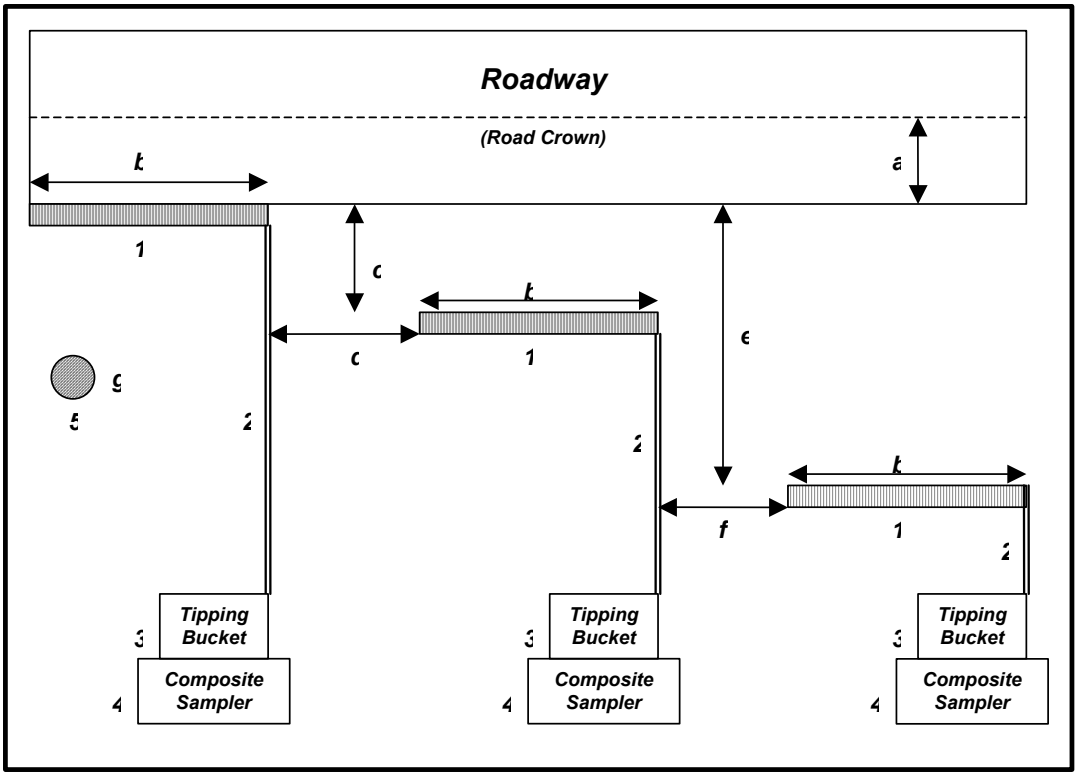

Figure 1. Remote Monitoring Site Layout

Table 2. Remote Monitoring Site Field Dimensions

Variable	Description	Roadway		
		Pullman SR 270	**Spokane US 2**	**North Bend I 90**
a	Distance from crown to edge of pavement	5.9m	8.6m	10m
b	Length of slot drain	2m	2m	2m
c	Distance from edge of pavement to 2nd slot drain	2.0m	3.2m	1.7m
d	Distance between 1st and 2nd slot drain	2.5m	3.0m	1.5m
e	Distance from edge of pavement to 3rd slot drain	4m	6.1m	3.5m
f	Distance between 2nd and 3rd slot drain	2.5m	3.0m	1.5m
g	Rain gauge height	3.1m	3.5m	~10m
Ksat	Saturated Hydraulic Conductivity (Sieve Test)	304 mm/hr	692 mm/hr	447 mm/hr

Measurement of natural rainfall at each site was performed by standard tipping bucket rain gauges equipped with recording data loggers (Hobo Event 4 – Onset Corp.). To determine the relationship between LID area (in this case lateral length from the edge of paved shoulder) and resulting runoff from the end of the pervious embankment, slot drains were installed at various distances down-slope from the roadway shoulder; typically 0m, 3.1m, and 6.1m (0ft, 10ft, and 20ft). Each slot drain was plumbed by PVC pipe to specially designed flow tipping meters (Figure 2). Tipping bucket flow meters have long been used as reliable, versatile method of measuring flow rates in runoff and erosion studies (Chow 1976; Edwards et al. 1974; Johnson 1942; Khan and Ong 1997). The use of tipping buckets was preferred over a critical flow measurement flume system due to concerns with debris clogging which could occur at the field locations. Each tipping bucket was calibrated in our laboratory to generate tip vs. volume relationships. Each flow meter was also equipped with similar recording data loggers as the rain gauges. The general site set-up is similar to another recent study by Caltrans to determine similar runoff relationships of roadway embankments (CALTRANS 2003). Photos of the remote sites are shown in Figure 3 through Figure 5.

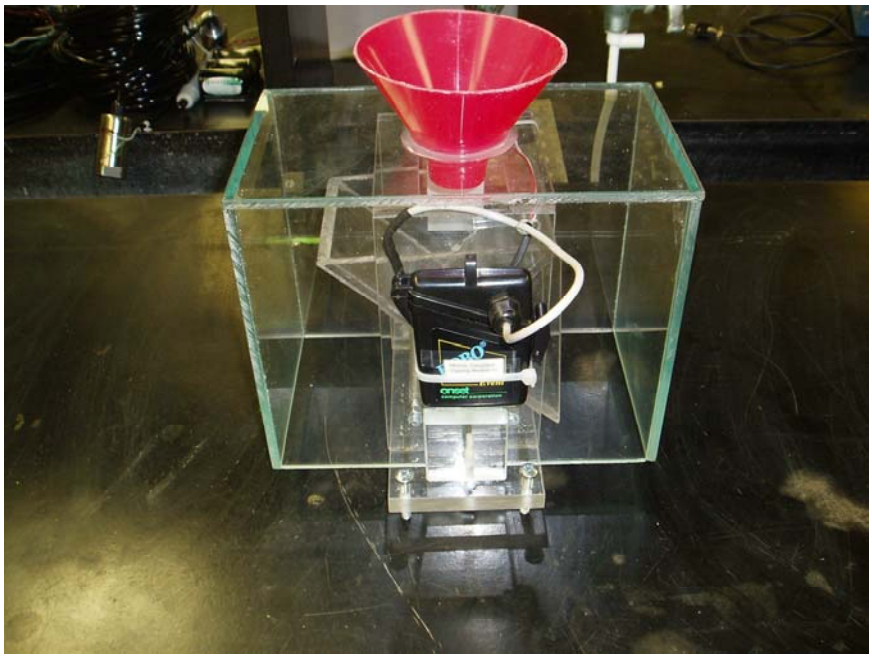

Figure 2. Tipping Bucket Flow Meter w/Onset (HOBO) Data Logger

Figure 3. US-2 Remote Monitoring Site (Spokane)

Figure 4. SR-270 Remote Monitoring Site (Pullman)

Figure 5. I-90 Remote Monitoring Site (North Bend)

Mobile Rainfall Simulation

The second portion of the data collection effort, vital to the numerical modeling component of the research, was rainfall/runoff simulation at various roadway locations. Mobile rainfall simulation included the application of controlled rainfall and run-on to LID area test plots of various lengths at different highway locations. By using a simulation method, variables included in the investigation of LID area runoff characteristics (rainfall intensity, contributory impervious area, slope, soil properties, and storm duration) could be easily controlled in the field and used as input parameters in the modeling phase; such control is not possible during natural rainfall events as studied in the static site monitoring. Rainfall simulation to determine infiltration and runoff characteristics of soil plots is a common investigative method found in the literature (Darboux et al. 2002; Fiedler et al. 2002; Frasier et al. 1995; Frasier et al. 1998c; Frasier et al. 1998a; Humphry et al. 2002; Pearce et al. 1998a; Springer 2004; Suleiman and Swartzendruber 2003; Weiler and Naef 2003). A run-on flow component was also included in the total hydrologic input of the study plots to simulate overland flow from upslope runoff (Frasier et al. 1998b; Pearce et al. 1998b). Differences between previous research and this effort are the use of rainfall simulation on constructed roadway embankments with the run-on component (to simulate roadway runoff) which is a much more significant portion of the total hydraulic input to the study plot.

Rainfall simulation was accomplished by the use of square-flow nozzles (FULLJET SS14WSQ, SS20WSQ, and SS50WSQ – Spraying Systems Co.). Single nozzles were mounted on a stationary boom and placed approximately 2.8m above the center of the test plots. Rainfall intensity during testing was measured by placing between 4 and 6 non-recording plastic rain gauges at each corner and center of the plots; depths were recording for given time durations

during simulation to calculate the average intensity. Both arithmetic and weighted means between the rain gauges were used to determine rain intensity used as an input parameter for computer modeling. Applied rain intensities were varied between 21 to 84 mm/hr (0.8 and 3.3 in/hr) and were adjusted by changing simulator nozzles, varying input flow rates (3.8 to 11.4 L/min), and changing the height of the nozzle above the plot (2.3 to 2.9 m). Water was supplied by a truck-mounted holding tank, which was in turn pumped through a rotometer manifold by an electric pump powered by a generator.

The run-on component was provided by a perforated PVC spreader-bar assembly placed at the uphill edge of the test plots. Water was evenly spread across the top of the plot and allowed to flow down slope to imitate dispersed flow originating from a roadway surface. Run-on flow rates were controlled through the same rotometer manifold system used for rain simulation. Flow rates were varied from 0 and 9.4 L/min (0 to 2.5 gal/min). As both the rain simulator and run-on simulator were supplied by the same storage tank (2270 L capacity) and pumping system, duration of testing (varying from 15 minutes to 1.5 hours) was limited based on total flow requirements and the volume of the storage tank.

Soil moisture measurements were made by installing 2 to 3 (depending on plot size) dielectric soil moisture probes (ECH_2O EC-20 – Decagon Devices, Inc.) into the soil plots at equidistant locations from the upstream plot edge. Soil moisture readings were taken at regular time intervals beginning at the start of each test. This information was used to study the propagation of the wetted soil front downhill in the test plots; equally, the initial soil moisture deficit was also calculated by taking the difference between the initial and saturated soil moisture readings. Initial soil moisture deficit, relative to saturation, was used as an input parameter for

the numerical modeling portion of the study, as discussed later. All tests were performed during Fall 2004; antecedent soil moisture contents were typically between 2-5% (relatively dry).

Test plots for mobile simulation were located on various highway embankments with slopes varying from 6:1 to 3:1. Plot sizes were also varied from 1m x 2m to 1m x 4m (3.3ft x 6.6ft to 3.3ft x 13.1ft) with the longest dimension in the down slope direction. The long sides of the plots were horizontally partitioned from the surrounding soils by inserting metal strips approximately 50mm (2in) into the ground. The top of the plot was equipped with the 1m (3.1ft) spreader-bar assembly and runoff was collected in a 1m (3.1ft) slot drain plumbed to a tipping bucket flow gauge discussed previously. Figure 6 and Figure 7 provide schematics of typical mobile rainfall simulation set-up. The photograph depicted in Figure 8 shows the field installation using a 4m (13.1 ft) plot located on US-395 while the photograph in Figure 9 shows a typical slot drain installation at the downstream end of the plot (similar to static site slot drain application).

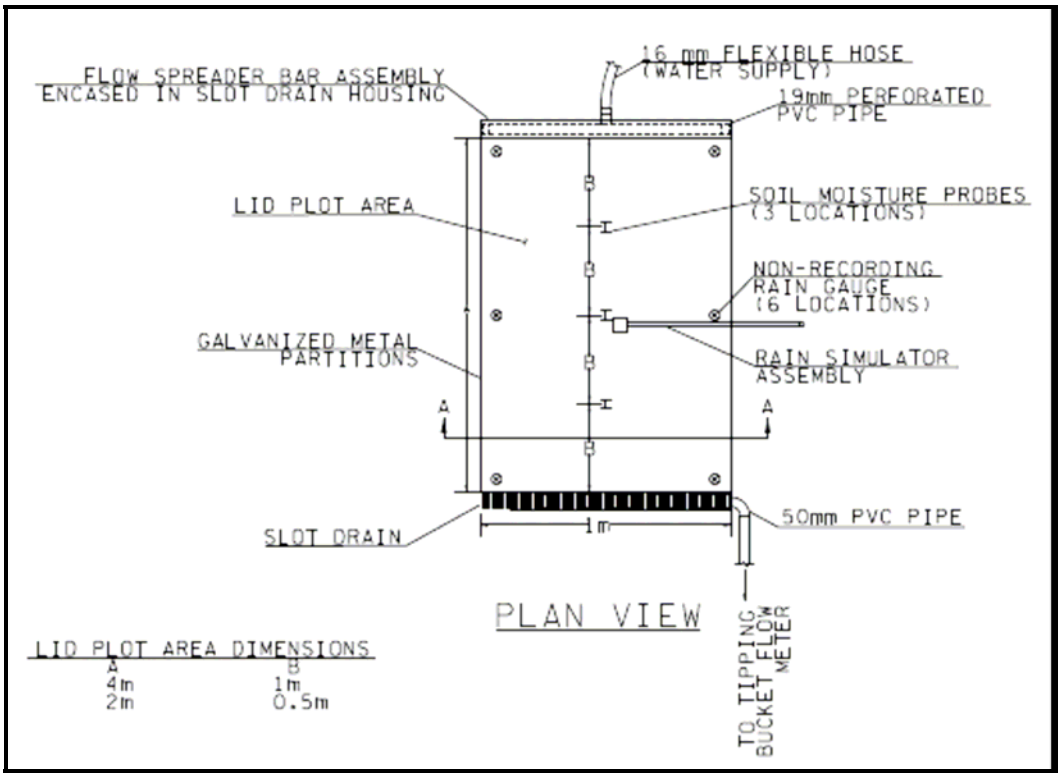

Figure 6. Mobile Simulation Schematic

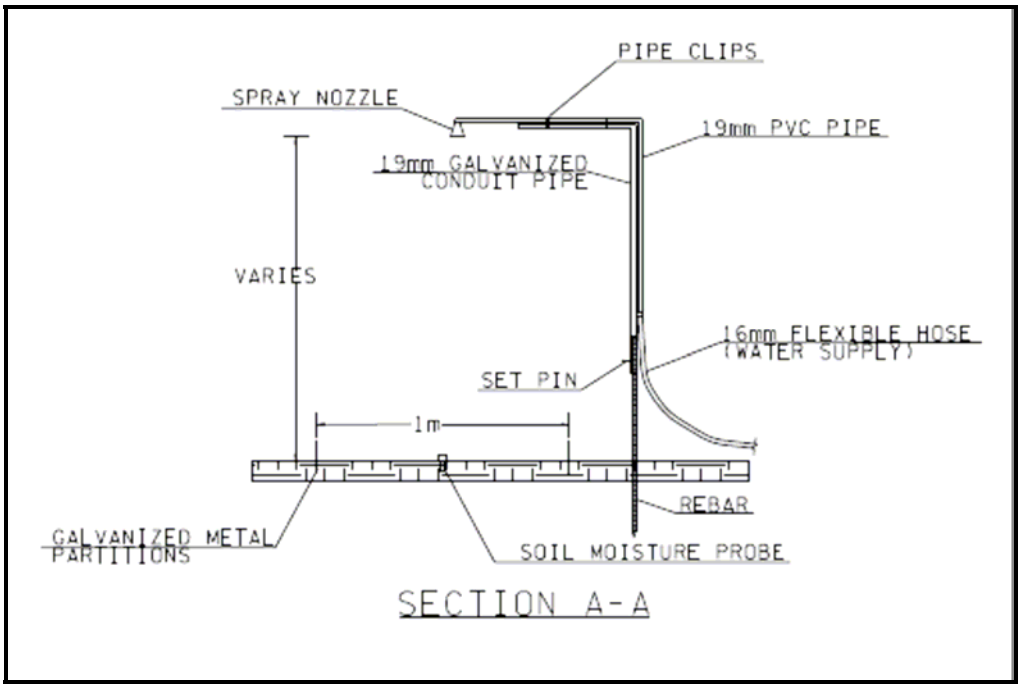

Figure 7. Mobile Simulation Schematic (Profile)

Testing protocol for mobile simulation involved application of a constant rain and run-on component for each test until steady-state runoff, defined by a relatively constant time interval between bucket tips, was achieved. Treatment (or infiltration) performance of each test plot under various rain/run-on conditions was then determined by calculating the portion of applied inflow that was measured as runoff at the end of the plot, which has been established as a well-accepted method of hydrologic analysis (Frasier et al. 1998d). The difference between applied rainfall (without a run-on component) and measured runoff, normalized over the total LID area, was used to estimate the hydraulic conductivity of the plot using the Equation 1:

$$K_e = r_i - q \qquad \textbf{(1)}$$

Where $\mathbf{K_e}$ = estimated saturated hydraulic conductivity (mm/hr), $\mathbf{r_i}$ = applied rainfall intensity (mm/hr), and **q** = measured normalized runoff intensity (mm/hr). Equation 1 is valid for steady-state, fully saturated conditions, which were achieved during field simulations. Normalizing the flow rate involves dividing the volumetric flow rate by the total effective plot area, allowing for easy comparison of runoff between plots of different sizes.

In addition to the simplified method for estimating K_s by Equation 1, soils samples from each simulation site (one per site) were collected and analyzed per the WSDOT's Simplified D_{10} and Modified D_{10} Sieve Analyses as described in the *Mobile Rainfall Simulation* section. A third method for K_s estimation, involving direct measurement with a Guelph Permeameter (Soilmoisture Equipment Corp.) was also used at each site.

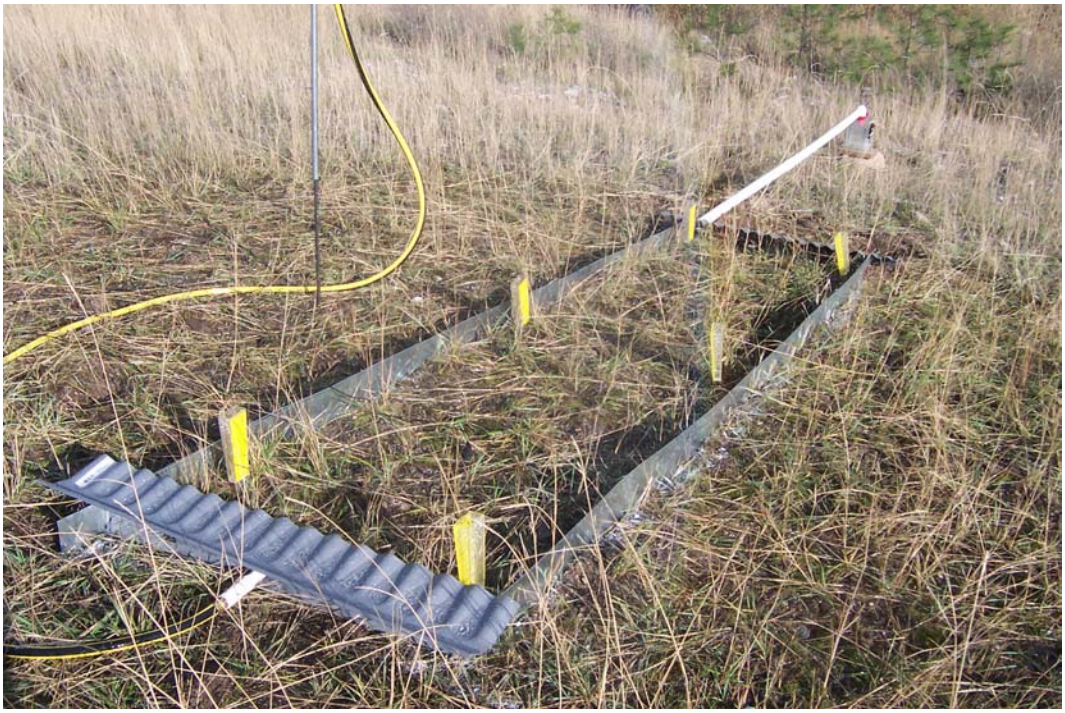

Figure 8. Mobile Simulation Plot (US-395)

Figure 9. Slot Drain Installation

Numerical Modeling

Consistent with WSDOT's need to adequately analyze existing natural dispersion guidance and propose new strategies, a physically based hydrologic flow model was necessary. Various numerical models have been proposed and effectively used to solve the complex process of rainfall infiltration with the runoff of excess water (beyond the infiltrative capacity of the soil) in hydrologic systems (Bronstert and Plate 1997; Castillo et al. 2003a; Corradini et al. 1998a; Deletic 2001f; Fiedler and Ramirez 2000; Galbiati and Savi 1995d; Howes and Abrahams 2003a; Leonard et al. 2004; Liu et al. 2004a; Munoz-Carpena et al. 1999c; Springer 2004; Wallach et al. 1997b). For the most part, approximations of the kinematic wave model using finite difference schemes have been used (Castillo et al. 2003b; Corradini et al. 1998b; Deletic 2001e; Galbiati and Savi 1995c; Howes and Abrahams 2003b; Liu et al. 2004b; Munoz-Carpena et al. 1999d; Springer 2004; Wallach et al. 1997a). As a majority of numerical methods developed are based on relatively continuous hydrologic conditions, where large discontinuities between soil properties (namely K_s) are not present, the kinematic approximation method is quite valid. However, given the large change in soil properties (most importantly K_s) and surface slope associated with highway runoff flowing onto highly permeable roadway embankments, a full hydrodynamic solution was sought for this study.

Fiedler and Ramirez Solution Approach

A numerical solution to the complete St. Venant equations, also known as the hydrodynamic wave equation (Chow et al. 1988), was proposed by Fiedler and Ramirez (2000). This model (MAC-2D) was chosen for this study due to its ability to handle the discontinuity condition in the saturated hydraulic conductivity (Ks) magnitude that arises between the impervious surface interface of the roadway edge and the pervious LID area as well as the run-on phenomenon. The model can also calculate overland flow wave propagation down slope with a near-zero depth

condition (i.e., a type of shock wave), a situation not easily approximated by kinematic wave models (Fiedler and Ramirez 2000).

MAC-2D is a finite difference solution to the full hydrodynamic wave equations for the overland flow component dynamically coupled with the Green-Ampt infiltration equation to describe the soil infiltration component. The Green-Ampt equation has been widely used by many authors for use in numerical models of infiltration of surface and rain waters into soil systems (Castillo et al. 2003c; Corradini et al. 1998c; Deletic 2001d; Galbiati and Savi 1995b; Liu et al. 2004c; Munoz-Carpena et al. 1999e; Springer 2004) and is described by Equation 2 (Chow et al. 1988):

$$F(t) = K_S \cdot t + \psi \cdot \Delta\theta \cdot \ln\left(1 + \frac{F(t)}{\psi \cdot \Delta\theta}\right) \qquad \textbf{(2)}$$

For this application of the Equation 2, $\mathbf{K_s}$ = saturated hydraulic conductivity [L/T], $\boldsymbol{\psi}$ = soil head suction potential [L], $\boldsymbol{\Delta\theta}$ = volumetric soil moisture deficit (L^3/L^3), **t** = elapsed time [T], and **F(t)** = time dependant cumulative infiltration [L]. Equation (2) can also be written in the form of instantaneous infiltration rate [L/T] at any time (t), as given in Equation 3 (Chow et al. 1988):

$$f(t) = K_S \cdot \left(\frac{\psi \cdot \Delta\theta}{F(t)} + 1\right) \qquad \textbf{(3)}$$

The Green-Ampt equation is a physically based solution to soil infiltration and its parameters are readily estimated by field measurements, which lend to its versatility in hydrologic field studies.

The Fiedler and Ramirez (2000) model has 7 input parameters that are used to solve the coupled hydrodynamic overland flow equations and the Green-Ampt infiltration equation. Model parameters include saturated hydraulic conductivity (Ks, cm/s), soil suction head potential

(ψ, cm), initial volumetric soil moisture content (θ, %), and the overland flow resistance friction parameter, K_0. The resistance friction parameter (K_0) is computed by Equation 4 (Fiedler and Ramirez 2000):

$$K_0 = f \cdot \mathrm{Re} \qquad \textbf{(4)}$$

Where ***f*** = the Darcy-Weisbach friction factor and **Re** = the Reynolds number, both well-known hydraulic parameters described in Chow et al (1988). (Note that Equation 4 is only valid in the laminar flow regime). Other model input values include a spatially discretionalized 3-D node system (surface model), applied rainfall intensity r_i (mm/hr), and duration of rainfall (sec).

In addition to normalized runoff hydrographs, the model also provides output for total cumulative infiltration F (cm), depth of overland flow H (cm), and overland flow velocities v (cm/s) at each x-y node. Although a full description of the model is beyond the scope of this paper, the reader is referred to Fiedler and Ramirez (2000) for further information and model verification. Model validation was presented in a later study, which indicated a high degree of model accuracy (Fiedler et al., 2002).

Coupling Mobile Simulation with Numerical Modeling

Of key importance in this study is the use of field rainfall/runoff simulation to calibrate and validate the numerical model for the highway systems currently being considered. This was achieved by fitting recorded field data from the mobile simulation phase with model outputs. Simulation plots were input into the spatial grid discretionalization of the model and field estimates of the K_s parameter were used as a starting point to calibrate the model output hydrograph. Of all input parameters, it was anticipated that the runoff hydrograph produced by the model would be most sensitive to the K_s parameter, as determined in previous studies (Castillo et al. 2003d; Deletic 2001c; Galbiati and Savi 1995a; Howes and Abrahams 2003c;

Springer 2004). A sensitivity analysis performed on the model for various non-simulated conditions confirmed this assumption.

Temporal and spatial discretionalization of the model, specifically the computational time-step interval and 3-D grid sizing (dx, dy, and z input) was established to satisfy the Courant condition as necessary in explicit finite difference schemes (Chow et al. 1988; Fiedler and Ramirez 2000). This resulted in a time increment (i) of 0.05 seconds, dx and dy lengths (cross and down slope directions, respectively) of 1cm, and spatially dependant z grid values based on surface slopes (50:1 slope used for roadway (impervious) surface with embankment slopes varying from 6:1 to 3:1). Larger time steps or larger x-y lengths produced instability in the model. Values of $K_{.s}$, ψ, and $\Delta\theta$ were assigned to each x-y node in the spatial grid and a constant slope angle for both the impervious roadway area and pervious LID area were applied (resulting in a smooth ground surface model). Although the model was written to handle total variability of hydrologic soil properties (K_s, $\Delta\theta$, ψ) and microtopographic features (z values) at each spatial node, to determine microtopographic and microhydrologic properties of the test plots was beyond the feasibility of this study. Therefore, lumped hydrologic properties and constant surface model slopes were used.

RESULTS AND DISCUSSION

Static Site Monitoring

Rainfall/runoff data from natural storm events were collected from September to December, 2004. In spite of the typical problems encountered during field sampling campaigns, including data logger failure and damage/clogging of tipping bucket flow meters, a total of 55 separate events were recorded (all sites combined). Site 1 had moderate precipitation activity with 17 recorded events and no apparent equipment failures. Site 2 on SR-270 had 36 rainfall events

successfully recorded. Although the most active site for rainfall was Site 3 on the western crest of the Cascade Mountains (I-90), the previously mentioned problems with monitoring equipment failure eliminated most hydrograph recording at this location and resulted in only 2 useable data events.

Original location and installation of the three remote sites was based on the assumption that significant runoff would be measured at short distances from the edge of paved shoulder with reduced runoff quantities positively correlated with increasing distance from the roadway edge. Vegetation was also expected to play a significant role in determining the relationship between slope length and measured runoff quantities. Observed data was far different, however. Site 1 (US-2) recorded only 5 events with any measurable runoff at the 3.0m distance and only 1 event recording flow at the 6.1m distance from the roadway edge. Runoff coefficients calculated for the 3.0m distance (evaluated by dividing the total normalized runoff depth by the related precipitation depth) never exceeded 0.2 for any of these events and usually averaged less than 0.1. The 6.1m distance runoff coefficient, again only calculated for the single measured event, was 0.08. Site 2 (SR-270) recorded no measurable runoff at any distance from the roadway edge for any event, even though there was no vegetation on the embankment slope. Early assumptions would dictate that Site 1 would outperform (yield higher infiltration rates of less down-slope runoff) Site 2 based on moderate vegetation and more shallow slopes; this was not the case, however. The most plausible explanation for this is the observed layer of fine silt/clay soils within 150mm (6 in) of the surface of the LID plots; this material effectively reduced the available soil water capacity at this site, which in turn reduced the effective infiltration rate for the longer duration events.

Site 3 was plagued by equipment problems; however, the two events successfully measured showed dramatic reductions in roadway runoff. No effective runoff was measured at either slot drain location away from the edge of paved shoulder.

Overall the static site monitoring method produced mixed results and was not a significant analysis method for this research, although it did provided a good "reality check" with the other two investigative methods, namely mobile rainfall simulation and numerical modeling.

Effects of Slope Angle on Runoff

Variability in site slopes included in the static site monitoring phase were limited due to site selection; therefore, comparisons between LID area slope and runoff are not conclusive; vegetative cover effect is the only considerable variation. However, a more expansive and similar research effort recently completed in the CALTRANS RVTS Study (2003), which focused on stormwater treatment performance of typical roadway embankments, suggests a lack of correlation between embankment slope angle and stormwater runoff attenuation (CALTRANS 2003). Results of the 2-year study are summarized in Table 3. Some sites included in the summary include impervious contributory areas not laterally contributing to adjacent to the LID areas, so effective unit roadway width lengths (as reported in Table 3) are normalized by dividing the total impervious area by the width of collection drains along the embankment slope, a convention used in this study and necessary for comparison between the two research efforts. Results from the RVTS Study (CALTRANS 2003) indicate that runoff coefficients (based on the ratio of total stormwater inflow to runoff volumes) are very low, even for 2:1 slopes. None of the 8 sites studied in this effort were engineered for stormwater treatment. The primary parameters correlated to stormwater attenuation/treatment performance of the road embankments were reported to be infiltration and vegetative cover (CALTRANS 2003).

Table 3. Caltrans RVTS Study (CALTRANS 20003).

SITE / SYSTEM		SLOPE (%)	Pavement Width (m)	LID Width (m)	ACP/LID Ratio	Vegetation (% cover)	Infiltration Rate (in/hr)	Relative Comaction (%)	Soil Type (USDA)	Runoff Coeff.
Sacramento	1	EP	16.1	0	NA	0	0.0		***D***	
	2	5%	16.2	1.1	14.7	93%	75.2	93.5%		0.31
	3	33%	16.6	4.6	3.6	84%	68.1	81.2%		0.32
	4	33%	17.8	6.6	2.7	92%	59.7	79.7%		0.28
	5	33%	17.4	8.4	2.1	90%	79.8	78.4%		0.15
Cottonwood	1	EP	10.5	0	NA	0%	0.0		***C***	
	2	52%	10.5	9.3	1.1	73%	88.9	85.8%		0.19
Redding	1	EP	9.0	0	NA	0%	0.0		***C***	
	2	10%	11.8	2.2	5.4	80%	48.0	93.0%		0.57
	3	10%	11.8	4.2	2.8	85%	84.8	88.6%		0.31
	4	10%	11.8	6.2	1.9	87%	105.4	78.8%		0.45
San Rafael	1	EP	61.6	0	NA	0%	0.0		***C/D***	
	2	50%	135.8	8.3	16.4	84%	236.0	78.8%		0.13
Yorba Linda	1	EP	30.3	0	NA	0%	0.0		***B***	
	2	14%	34.2	1.9	18.0	61%	32.0	89.2%		0.37
	3	14%	33.8	4.9	6.9	82%	22.1	82.5%		0.51
	4	14%	28.7	7.6	3.8	74%	39.9	87.7%		0.58
	5	14%	48.4	13.0	3.7	76%	46.0	86.8%		0.17
Irvine	1	EP	19.4	0	NA	0%	0.0		***B***	
	2	11%	22.6	3.0	7.5	70%	39.1	88.4%		0.39
	3	11%	22.6	6.0	3.8	63%	41.9	84.7%		0.05
	4	11%	32.9	13.0	2.5	62%	23.4	87.6%		0.16
Moreno Valley	1	EP	15.4	0	NA	0%	0.0		***C***	
	2	13%	16.6	2.6	6.4	3%	18.3	90.7%		0.95
	3	13%	16.8	4.9	3.4	16%	14.5	93.3%		0.95
	4	13%	17.4	8.0	2.2	22%	23.9	92.9%		0.48
	5	13%	18.0	9.9	1.8	18%	26.4	93.9%		0.51
San Onofre	1	EP	26.6	0	NA	0%	0.0		***B/C***	
	2	8%	22.0	1.3	16.9	81%	57.2	95.9%		0.45
	3	10%	23.9	5.3	4.5	74%	31.8	88.5%		0.27
	4	16%	25.1	9.9	2.5	69%	19.1	85.3%		0.07

The primary runoff attenuation parameter was the hydraulic conductivity of the soil, regardless of slope. For more information on the previous research and methods used therein, the reader is directed to CALTRANS (2003).

Mobile Rainfall Simulation

A total of 16 simulated storm events were completed during this phase of the research, including both rainfall/run-on and rainfall-only tests. Summaries of field simulations are reported in Table 4. It was observed that effective infiltration rates associated with higher rainfall/run-on intensities were consistently higher than those estimated for the lower intensity tests using Equation 1; this phenomenon, generally identified as “interactive infiltration”, is well documented in the literature (Corradini et al. 1998d; Fiedler et al. 2002; Morin and Kosovsky

1995; Springer 2004). Interactive infiltration results from spatial variability of the physical properties of the plot, most specifically areas of variable hydraulic conductivity magnitudes and microtopography. As runoff propagates down slope, it may be intercepted by areas of localized higher K_s values, effectively infiltrating a large percentage of the overland flow without the contribution of the total surface plot area for runoff. Smaller runoff flow rates resulting from rainfall and impervious runoff may also concentrate as overland flow in microtopographic low areas void of vegetation and areas with other macropore features (Fiedler et al. 2002; Howes and Abrahams 2003d; Weiler and Naef 2003), hence lowering the apparent effective hydraulic conductivity of the system. As rainfall and run-on increases, the depth of overland flow in these microtopographic regions can increase, or areas that were fully infiltrating the excess runoff previously may become saturated and begin to contribute to runoff; this process causes overland flow to move into areas of higher hydraulic conductivities not previously saturated, thus increasing the apparent effective infiltration of the entire test plot (Corradini et al. 1998e; Fiedler et al. 2002; Morin and Kosovsky 1995; Weiler and Naef 2003). As mentioned previously, estimating individual node hydrologic soil properties was outside the scope of this study due to practical limitations; therefore spatially averaged lumped K_s values were used to describe the entire test plot – giving rise to variable infiltration rates calculated for varying rainfall intensities. Estimation of K_s values was made initially by performing rain-only simulations on the test plots by using Equation 1, with values reported in the "Ks Test" column of Table 4. Soils samples (one per test location) were also collected and sieve analyses were performed per current WSDOT guidelines and Massman (2003). Both the Simplified D_{10} and Modified D_{10} sieve analyses (Massman 2003) methods were used; results of these tests are reported in Table 4 ("D_{10}" and "Modified D_{10}" columns, respectively). ASTM D422 Gradation Testing is used for the

Simplified D_{10} test, with recommended long-term infiltration rates presented in Table 5 (WSDOT 2004b). The Modified D_{10} Analysis is a more detailed approach, with K_s values calculated using Equation 5 (Massman 2003; WSDOT 2004b):

$$\log(K_S) = -1.57 + 1.90 \cdot D_{10} + 0.015 \cdot D_{60} - 0.013 \cdot D_{90} - 2.08 f_{fines} \quad \textbf{(5)}$$

Where D_{10} ,D_{60} and D_{90} are the grain sizes in mm for which 10, 60, and 90 percent of the sample is finer and Df_{fines} is the fraction of the soil by weight passing the #200 sieve (Massman 2003; WSDOT 2004b). Units for K_s in Equation 5 are given in feet/day.

Table 4. Mobile Simulation Test Summary

Test # (Mobile #)	Site	LID Lenth (m)	LID Width (m)	LID Slope (m/m)	Test Duration (min)	Rain (mean) mm/hr	Rain (Dev) (mm/hr)	Rain (wmean) (mm/hr)	Runon (L/min)	ACP Length (m)	ACP/LID Ratio	Max Runoff (mm/hr)	Runoff %	Time to Peak (min)	Ks Test (mm/hr)	Ks for Model (mm/hr)	Modified D_{10} Ks (mm/hr)	Modified D_{10} Ks (mm/hr)	GP (mm/hr)
1 (1)	US 2	1.95	1.03	6:1	30	25.4	*	*	3.8	8.68	4.5	24.2	95.2%	15	_	9.0	30.5	706.5	104.2
2 (1)	US 2	1.95	1.02	6:1	58	25.4	*	*	3.8	8.77	4.5	15.8	62.2%	45	_	54.0	30.5	706.5	104.2
3 (2)	US 2	1.92	1.02	6:1	68	25.4	*	*	3.8	8.77	4.6	10.8	42.4%	**	_	82.8	30.5	706.5	104.2
4 (3)	US 2	4.03	1.02	6:1	15	25.4	*	*	3.8	8.77	2.2	17.5	68.9%	5	_	_	30.5	706.5	104.2
5 (4)	US 2	4.03	1.03	6:1	30	45.7	30.0	65.3	3.8	3.38	0.8	31.0	47.6%	26	35.3	57.6	30.5	706.5	104.2
6 (6)	US 395	4.00	1.05	4:1	69	27.9	11.9	26.5	4.9	10.59	2.6	10.0	37.5%	37	42.2	63.0	27.9	248.9	-
7 (6)	US 395	4.00	1.05	4:1	20	67.7	23.5	72.4	0.0	0	_	42.7	63.1%	10	17.4	39.6	27.9	248.9	-
8 (7)	US 395	4.00	1.05	4:1	45	32.2	14.2	29.8	4.9	9.42	2.4	11.2	37.5%	36	21.9	52.2	27.9	248.9	-
9 (7)	US 395	4.00	1.05	4:1	25	80.3	10.4	85.6	7.6	5.39	1.3	20.7	25.8%	16	21.9	153.0	27.9	248.9	-
10 (8)	US 395	4.00	1.05	4:1	42	35.2	11.5	32.1	4.9	8.77	2.2	14.8	46.3%	36	21.9	50.4	27.9	248.9	-
11 (9)	US 395	3.90	1.00	4:1	70	37.9	12.4	33.9	4.9	7.78	2.0	0.0	0.0%	N/A	_	_	114.3	632.5	-
12 (10)	US 395	4.00	1.02	4:1	30	70.9	11.5	72.0	9.5	7.86	2.0	0.2	0.2%	15	_	_	114.3	632.5	-
13 (10)	US 395	2.00	1.02	4:1	30	66.7	1.3	_	9.5	8.35	4.2	1.6	2.3%	5	_	333.0	114.3	632.5	-
14 (11)	US 195	4.00	1.02	3:1	60	32.0	4.0	32.9	4.9	8.80	2.2	0.2	0.5%	24	76.6	_	22.9	566.4	101.8
15 (12)	US 195	2.00	1.01	3:1	52	0.0	0.0	_	4.9	14.12	7.1	0.0	0.0%	N/A	66.7	_	22.9	566.4	101.8
16 (13)	US 195	2.00	1.01	3:1	40	0.0	0.0	_	9.46	6.68	3.3	39.8	47.3%	25	66.7	180.0	22.9	566.4	101.8

It is apparent from the values in Table 4 that the Simplified D_{10} test values for K_s greatly under predicted the Modified D_{10} results by at least an order of magnitude. Apparent effective K_s values determined by Equation 1 and also by model calibration (reported in "K_s for Model" column, Table 4) were consistently higher than the Simplified D_{10} and significantly lower than the Modified D_{10} results. Factors affecting the large disparity between the sieve analysis results that yielded the Simplified D_{10} values can be attributed to the presence of large cobbles in lower soils layers that were observed in the soil samples. Large cobbles affect the Modified D_{10} method (increasing it) but do not impact the Simplified D_{10} results. Better sampling methods of the uppermost soil layer, which were difficult due to the presence of vegetation, would likely reduce the variability of these results. Note, however, that grain size infiltration methods do not account for packing (variable soil density) or small-scale layering, and are therefore likely not the best method for determination of hydraulic conductivity values in surface infiltration studies, especially in highly compacted soils. These factors increase the calculated infiltration rates of the soils beyond what is actually found in the field.

Table 5. Recommended Infiltration Rates Based on ASTM D422 (WSDOT 2004)

D_{10} Size from ASTM D422 Soil Gradation Test (mm)	Estimated Long-Term (Design) Infiltration Rate (inch/hour)
≥ 0.4	9
0.3	6.5
0.2	3.5
0.1	2.0
0.05	0.8

Direct K_s measurements were also taken using a Guelph Permeameter (Soilmoisture Equipment Corp), with results reported in Table 4 under the "GP" column. No data were obtained at the US 395 sites as soil conditions prevented successful test well auguring necessary for the standardized test. The Guelph Permeameter matched the modeled K_s values more closely

than the Modified D_{10} method. Regardless, estimation of K_s values is generally difficult and can vary several orders of magnitude depending on sample location and testing methods (Freeze and Cherry 1979) and remains a challenging obstacle in the design of surface infiltration mechanisms such as natural dispersion.

Effects of Slope Angle on Runoff

In general there was no discernable performance differences over the range of slopes evaluated in this study. In fact, the LID areas on the 3:1 and 4:1 slopes outperformed the 6:1 slope, which is much closer to the current WSDOT natural dispersion guidelines for acceptable slope (7:1). This can be attributed to the relatively high microtopographic and microporous characteristics of the test plots.

As stated previously, current design guidance for the use of natural dispersion uses LID area slope rate as a threshold parameter that results in the elimination of certain areas for incorporation as a treatment area or dramatically increases the required length of the LID area. This guidance is not supported by the results of this study; in actually, the most critical design parameter appears to be the infiltrative capacity (and thus K_s) of the soil of the LID area. Similar conclusions that increasing plot slope angle does not necessitate increased runoff have been previously reported in the literature (Barrett et al. 1998c; Deletic 2001b; Joel et al. 2002b; Munoz-Carpena et al. 1999f). Although it is apparent that existing natural dispersion guidance is based on slopes analyzed for stormwater quality control (which focuses on sediment deposition in roadside grass filter strips), Delectic (2001) explained that increased retention time of runoff water on slopes of shallow angles is only important if the soil infiltrative capacity is capable of absorbing the excess runoff during the increased retention time; where soil moisture capacities are large, slope has little effect on runoff (Deletic 2001a). It follows that if full dispersion of overland flow is maintained on embankment slopes, increasing the slope for the range of slopes

evaluated does not decrease the area available for infiltration of that runoff. Additionally, under steady-state saturated soil conditions (as simulated in this research), the key parameter for estimating runoff is K_s.

Although erosion due to flow concentration is considered to be a key drawback in using steeper slopes for runoff treatment, the process involved in sediment transport (erosion) is based on numerous factors, including soil grain size, flow regime and velocity, and slope (Yang 2003); but not slope alone. Although outside the scope of this study, the effects of sediment transport and erosion on various slope angles were not apparent over the range of slopes studied. In fact, no flow concentration or detectable erosion was observed for any simulation test and no erosion was observed at the static site locations where flow concentration did not occur at the pavement edge, even for embankments with minimal vegetation.

Numerical Modeling

Calibration with Mobile Rain Simulation Results

Parameters estimated during field simulations were used as input variables into the Fiedler and Ramirez (2000) 2-D hydrologic model. In general, the estimated K_s values determined by rain-only simulations and Equation 1 showed good agreement with the predictive model output for steady-state runoff. Other parameters, such as the friction factor ($K_{0)}$ and initial soil moisture deficit ($\Delta\theta$) were also used to provide a better fit between the measured and simulated runoff hydrograph. Soil moisture deficit was calculated as described in the Experimental Methods Section. Again, K_s was the most sensitive and effective source parameter used to calibrate the model for both time to runoff, time to peak, and steady-state runoff intensity.

Impervious contributory surface length (unit width) values used in the model were determined by assuming run-on volumes applied in the field were the result of 100% runoff of the simulated rainfall intensity from the roadway. By dividing the applied run-on flow rate

[L^3/T] by the applied rainfall [L/T] and unit width [L^2], an associated impervious roadway length was determined [L]. All impervious roadway surfaces were input with 2% slopes, typical for most highway configurations (WSDOT 2004a). LID area geometry was input as measured in the field.

Results of two calibration model runs are shown in Figure 10 and Figure 11. Due to the simplifications used in the modeling, specifically a uniform LID area slope (without microtopography) and lumped uniform K_s values, time to runoff and time to peak characteristics of the measured runoff hydrographs were the most difficult aspects to duplicate. As mentioned previously, microtopographic features which are responsible for surface storage and interactive infiltration have pronounced effects on runoff. However, since the primary goal of the study was to determine *steady-state* runoff intensities, these aspects were not considered critical to the analysis. As a future research effort, the direct effects of interactive infiltration and microtopography on natural dispersion in the highway environment should be pursued.

Several field rainfall simulations produced no measurable runoff from the LID plots under rain-only events, even though applied intensities (21 to 84mm/hr) far exceeding typical moderate duration storm events. With no runoff data generated under rain-only conditions, calibration based on K_s on these plots was only possible when a run-on (highway runoff) component was included in the field rainfall simulation.

Simplified Modeling Procedure

Use of the Fiedler and Ramirez (2000) numerical model produced accurate steady-state runoff intensities related to measured field simulations (Figure 10 and Figure 11). While a valuable tool in the analysis of natural dispersion performance of roadway embankments, the model is somewhat complicated and computationally intensive and may not be a viable method

for widespread use by highway designers. A simplified equation that estimates natural dispersion design parameters given known (or measurable) variables is desired.

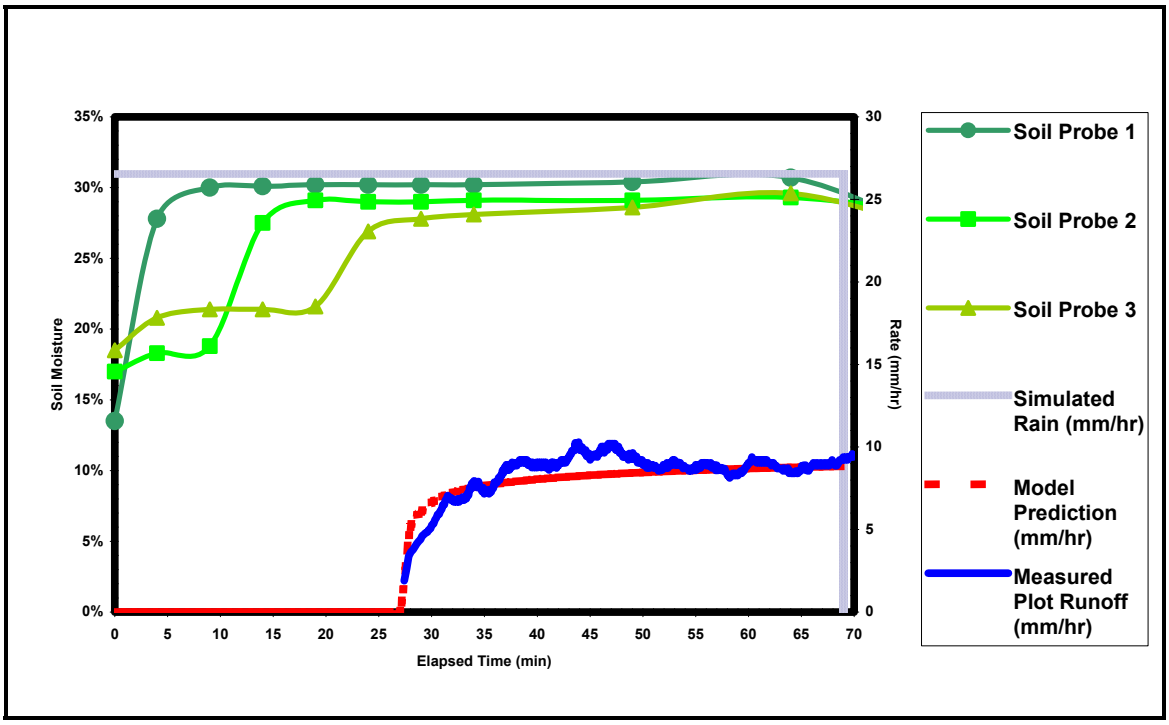

Figure 10. Rainfall/Runoff Hydrograph - Test 6

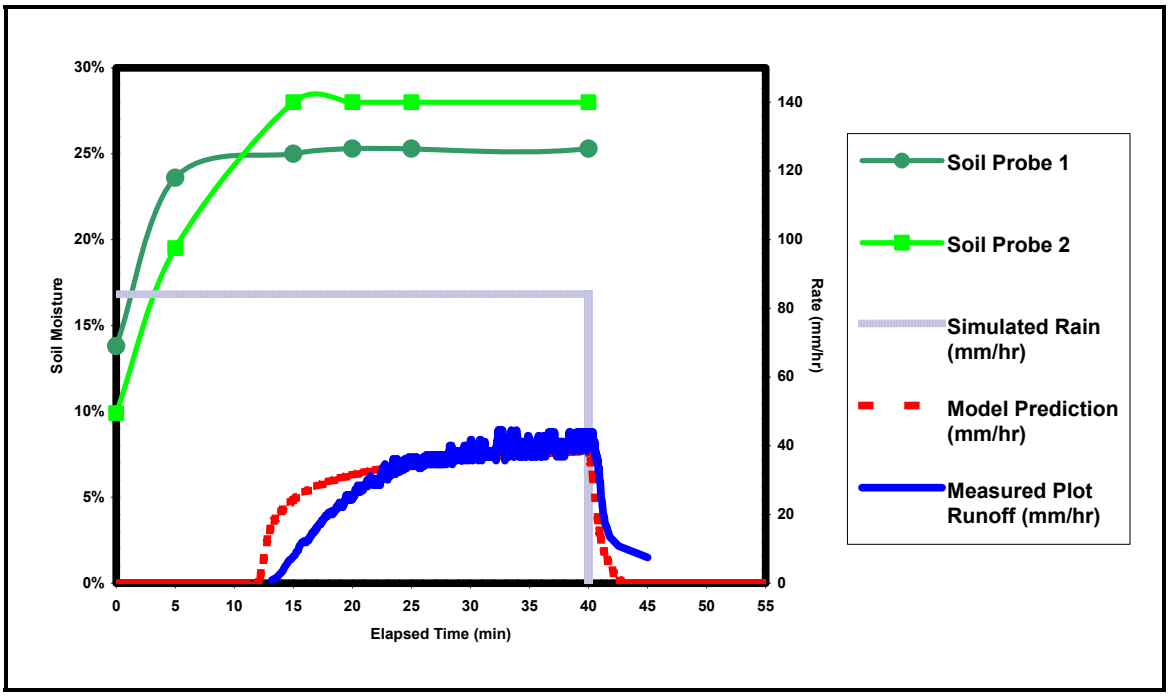

Figure 11. Rainfall/Runoff Hydrograph - Test 16

Current natural dispersion guidelines are not site specific. Additionally, the inclusion of a design rainfall intensity, which varies depending on geographic location for other BMP strategies, is not considered for natural dispersion. This seems to be a significant oversight in the current guidance as highway facilities in the Columbia Basin receive much less precipitation than a majority of western Washington or mountainous locations. Based on this fact, together with the variability of soils throughout the state, a "one size fits all" design approach not based on sound engineering analysis.

Natural Dispersion Design Equation

Typically soil K_s values, roadway width, and embankment slope are known (or measurable) design values. Also, tabulated design storm intensities have been published for all geographic areas in Washington State (WSDOT 2004b). Basing natural dispersion design on these key variables would seem a more responsible and accurate approach than current guidelines. As

shown previously, the most influential physical properties associated with natural dispersion are contributory unit roadway width (hereafter termed "ACP", [L]), rainfall intensity (r_i), and saturated hydraulic conductivity (K_s). The unknown variable to be determined is the length of LID area [L] perpendicular to the roadway. Applying these variables into a simplified flow balance results in Equation 6:

$$\frac{ACP \cdot r_i + LID \cdot (r_i - K_s)}{(ACP + LID)} = q \qquad \textbf{(6)}$$

Where **ACP** and **LID** are lengths of unit width [L^2] with all other variables defined previously. Knowing that the purpose of natural dispersion is to infiltrate all rain and highway runoff, **q** can be set = 0 and Equation 6 can be rearranged to solve for the design parameter **LID** in the form of Equation 7 (hereafter referred to as the LID design equation):

$$LID = \frac{ACP}{\frac{K_s}{r_i} - 1} \qquad \textbf{(7)}$$

Equation 7 is based on the unit width principle applied to this analysis; therefore, since both the **ACP** and **LID** variables have units of [L^2] and are the same width (assumed 1m), by dividing by the unit width we can reduce the units of each variable to that of [L], consistent with the variables being considered. Equally, Equation 7 assumes fully saturated soil conditions with a hydraulic gradient equal to unity (gravity flow).

Multiple simulations using the Fiedler and Ramirez (2000) model were run using various combinations of impervious surface length (ACP), LID area length (LID), K_s, and r_i. Model output was compared to the results of the simplified Equation 6 using the same input variables. Agreement between the two methods was very good, as shown in Figure 12. With all

combinations of realistic field conditions simulated, the equation consistently predicted the resulting runoff to within 6% of that determined by the computer model, although in all cases the differences should be considered minor and is most likely the result of modeling simplifications. Equation 7 is therefore a valid estimator of appropriate LID area length given other site-specific variables. If desired, further conservatism could be applied to Equation 7 by inclusion of a correction factor based on the potential for subsurface return flow as observed in other studies. However, since the use of K_s as the primary infiltration parameter is included in Equation 7, it should be noted that the situation represented by the equation is, in all reality, a very conservative method of analysis.

Determination of design rainfall intensity is critical for the implementation of Equation 7 for natural dispersion design. Since the equation includes only a single intensity value, the basis

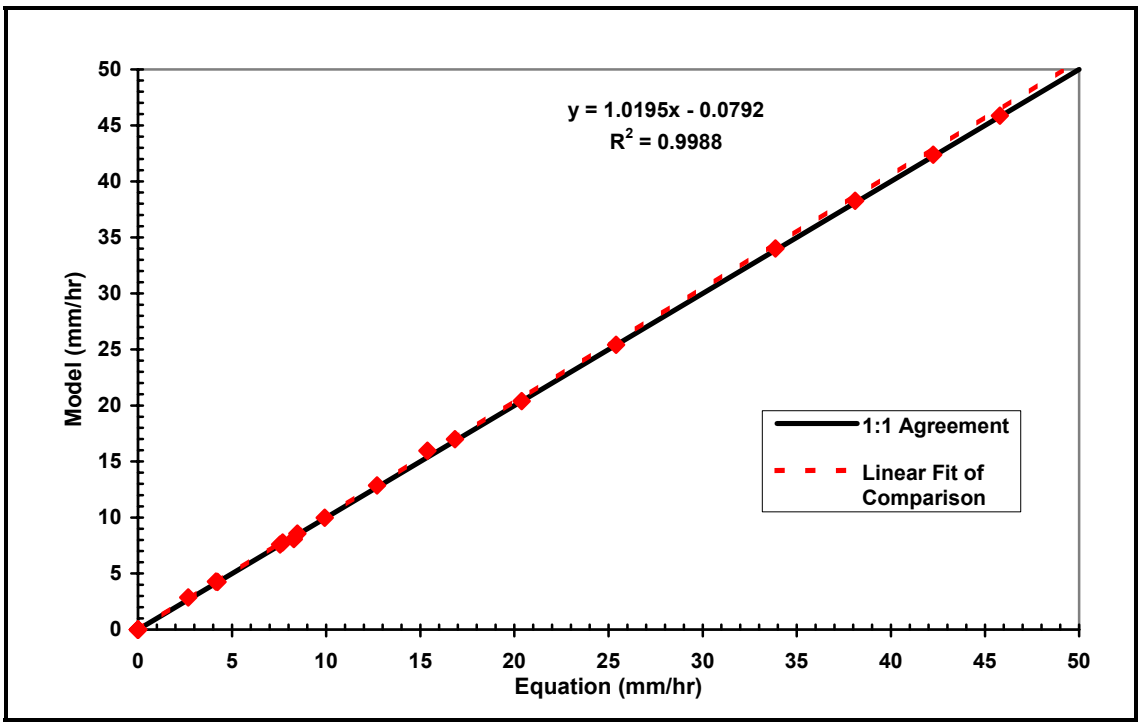

Figure 12. Comparison of Model and Equation 6 for Steady-State Runoff

for its use is from a maximum flow rate standpoint. Therefore, if the peak design rainfall intensity for a given site can be attenuated, all subsequent lower intensities will also be infiltrated in the LID area as long as the embankment soil has the capacity to infiltrate the entire volume. For the same duration, all modeling simulations used to evaluate Equations 6 and 7 assumed fully saturated conditions, therefore K_s does in fact equal the long-term infiltration rate (Chow et al. 1988). However, if hydraulic gradients are reduced below unity due to water mounding in the vadose zone, resulting in a lack of soil water capacity, the actual infiltration rate will be reduced below K_s based on Darcy's Law (Freeze and Cherry 1979; Massman 2003).

The dry soil depth (**L**), necessary to infiltrate the total cumulative infiltrated water depth (**F**) is related to the porosity of the soil (equal to the saturated soil water content) (Freeze and Cherry 1979) and is given by Equation 8 (Chow et al. 1988):

$$L = \frac{F}{\theta_{sat}} \qquad \textbf{(8)}$$

Where θ_{sat} = saturated volumetric soil water content. For soils with a residual water content, $\Delta\theta$ can be used in place of θ_{sat} in Equation 8 to estimate the necessary soil depth. The Fiedler and Ramirez (2000) model provides cumulative infiltration depth as an output. Although a simple single intensity storm event has a predictable relationship between distance from roadway edge and cumulative infiltration (Figure 13), for multiple intensity storms this relationship is quite complicated and requires a full storm analysis. One such numerical simulation was performed using the long-duration design storm hyetograph provided for BMP quantity control design from WSDOE (2004). The results of the model simulation, using a total 36 hour precipitation depth of 27mm with ACP and LID lengths of 6.1m and 3.1m, respectively, is

shown in Figure 14. As no runoff resulted from this configuration, the depth of infiltration at the very downstream end of the LID plot is equal to the total rainfall depth.

For a majority of the mobile simulations performed in this study, the saturated soil moisture content was approximately 30%. Using Equation 8 and the total depth of infiltrated stormwater from Figure 11 (135 cm), it is easy to determine that the soil depth required (with the given properties) must equal at least 4.5m in the vertical direction below the embankment surface at the edge of pavement to fully infiltrate the runoff. Of course this required depth decreases rapidly with distance from the edge of pavement; on highway embankments, the depth of fill material incorporated into the embankment also decreases with distance from the roadway. The model cannot address the effects of not achieving the required depth of infiltration. Therefore further study or modeling techniques are necessary to fully answer questions relating to soil depth requirements for full infiltration of runoff. It may be realistic, however, to require a necessary soil depth equal to the total depth of runoff evenly distributed across the length of the LID area after applying Equation 8.

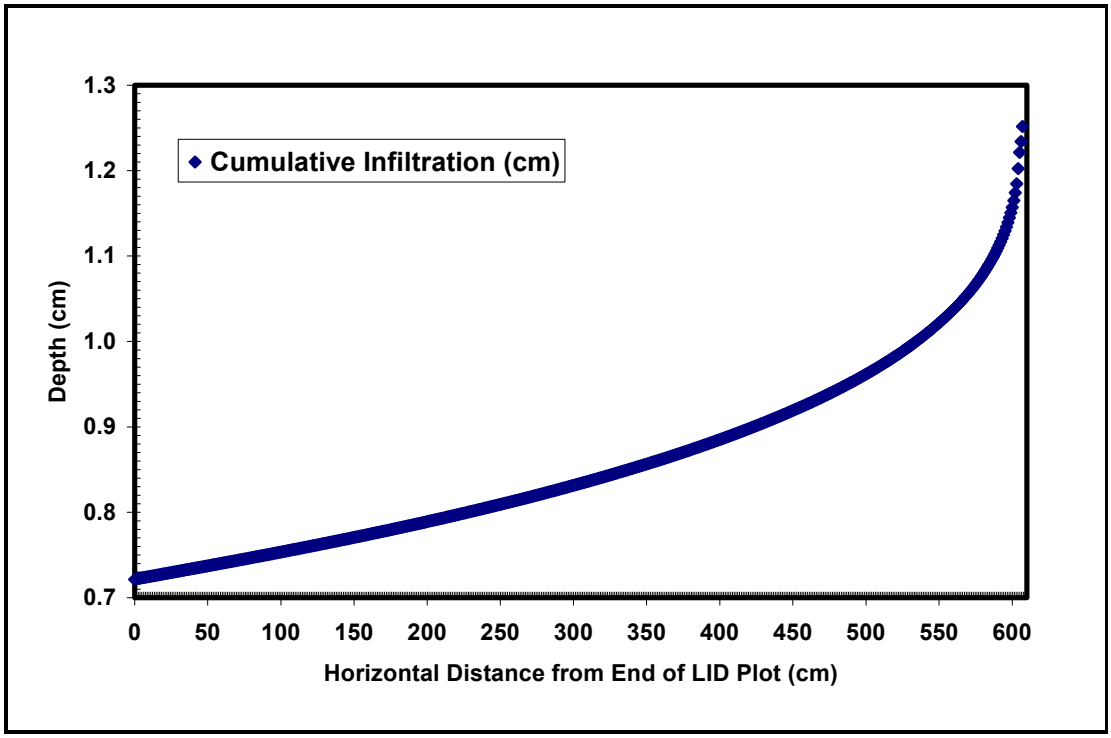

Figure 13. Cumulative Infiltration Across LID Length (Single-Intensity 30 Minute Event)

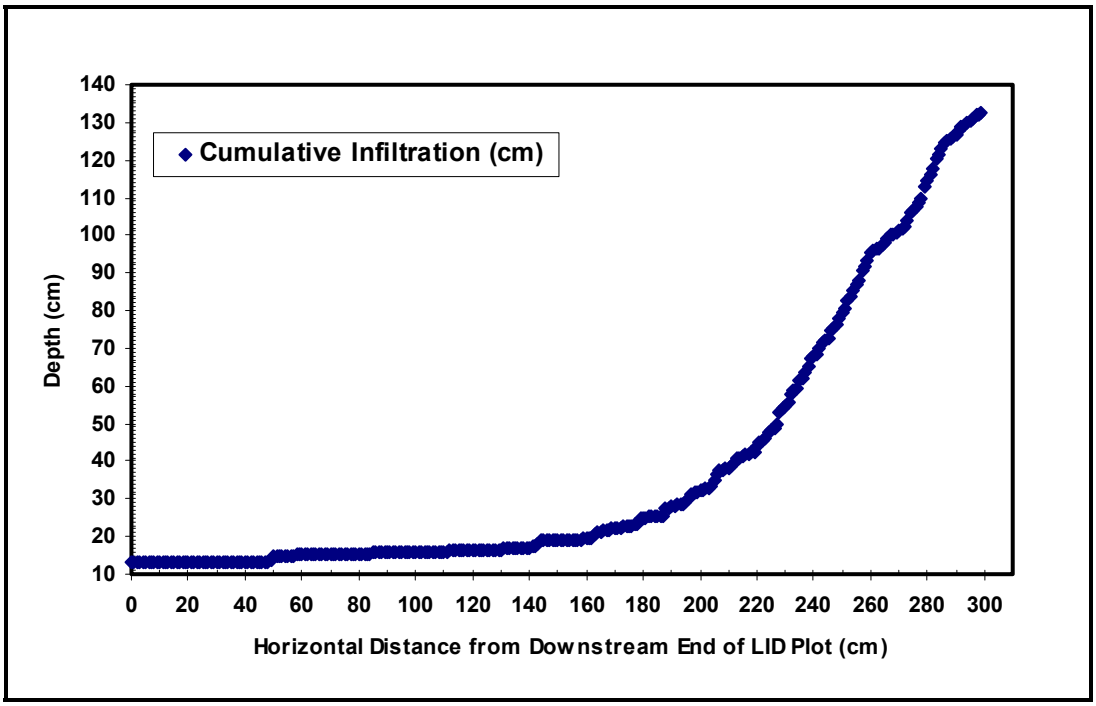

Figure 14. Modeled Cumulative Infiltration Across LID Length (36-Hour Storm)

Evaluation of Existing Regulatory Guidelines

Existing guidelines for natural dispersion design were evaluated using Equation 7. Although the current guidance is not based on design rainfall intensities (a spatial/temporal variable) or variable Ks values, an evaluation of LID scenarios based on the peak 5 minute intensity of the semi-annual 3-hour short-duration thunderstorm (WSDOT 2004b) reveals the current WSDOT guidance varies significantly based on highway location and design variables. This design storm event was chosen based on its high peak intensity and use as the flow-based BMP quality treatment storm event; longer duration quantity treatment design storm events have lower peak intensities. Therefore in a steady-state analysis, if the higher intensity quality treatment design storm is infiltrated, it follows that lower intensity quantity treatment events are also infiltrated within the same LID area length.

Various scenarios to contrast current LID requirements (regardless of embankment slope) against Equation 7 (using the previously selected 3-hour short-duration storm intensity) are shown in Table 6; assumed embankment Ks values have been assigned to illustrate the disparity between the examples. As depicted in Table 7, "N/A" denotes that due to Ks values below the required 102 mm/hr, current WSDOT LID guidance suggests these situations are not appropriate for the use of natural dispersion, although relatively short LID dimensions would satisfy Equation 7. Similarly, where site Ks values exceed the minimum requirement, no reduction in LID length is allowed based on current guidance.

For highway locations high in the Cascade Mountains, such as at Stevens Pass, existing guidance indicates a much shorter LID length as compared to Equation 7 (see TABLE 7). This is due to the higher peak storm intensities found in mountainous areas. For areas such as the Columbia Basin, where peak storm intensities are much smaller than those in the mountains, Equation 7 indicates that existing guidance is too conservative at locations where Ks exceeds

102mm/hr. Unfortunately, should Ks values be anything less than the current minimum value of 102mm/hr, natural dispersion would not be an accepted method of treatment at these locations, even though ample dispersion lengths may be available. Equally, the maximum slope requirement of 7:1 in current guidelines would preclude accounting for any natural dispersion benefit from highway embankments, which (as previously stated) are typically designed to be built at slopes ranging from 6:1 (flattest) to 2:1 (steepest). As shown by this and other studies, for the range of Ks values included, slope angle has little effect on infiltration rates as long as runoff is unconcentrated.

Table 6. Current LID Application Requirements vs. Equation 7

Highway Location	Roadway Width (m)	Embankment K_{sat} Value (mm/hr)	Current WSDOT LID Length Guidance (m)	Equation 7 LID Length Requirement (m)
Stevens Pass	14.0	152	5.0	8.0
Stevens Pass	14.0	102	5.0	16.6
Stevens Pass	6.1	102	3.0	7.2
Wenatchee	14.0	152	5.0	2.4
Wenatchee	14.0	75	N/A	5.9
Wenatchee	6.1	75	N/A	2.6
Moses Lake	14.0	152	5.0	2.1
Moses Lake	14.0	102	5.0	3.4
Moses Lake	6.1	50	N/A	4.0
Spokane	14.0	152	5.0	2.9
Spokane	14.0	102	5.0	4.7
Spokane	6.1	102	3.0	2.1

By basing LID lengths on available infiltration values and temporal rainfall intensities, using Equation 7 shows the existing guidance to be inaccurate in prescribing required dispersion lengths, especially for scenarios where the Ks/ri ratios approach unity and where Ks values are less than currently required (102 mm/hr) where natural dispersion is not allowed. It is apparent that for situations that dominate rural highway settings, particularly those in eastern Washington, such as low storm peak intensities and 2-lane highways, natural dispersion should be applied based on the results of Equation 7. Equally, requiring LID area slopes to be no steeper than 7:1 is not supported by the current research.

CONCLUSIONS

Effective stormwater quantity treatment remains an ongoing challenge for highway engineers due to the expansive linear nature of non-urban roadways. Centralized treatment strategies, such as wet-ponds and infiltration basins require containment and conveyance infrastructure to function effectively. Equally, significant real estate requirements and routine maintenance needs make these BMPs difficult to implement. New concepts in stormwater treatment known as *low impact development*, which focus on decentralized strategies such as natural dispersion, may be a viable solution.

Natural dispersion seeks to treat highway stormwater runoff by allowing non-concentrated flows to infiltrate into pervious roadside areas without the need for conveyance or detention facilities. However, current design guidelines for such measures are based on existing stormwater quality BMP methodologies and have previously not been investigated as to their suitability; in some cases the resulting requirements are too conservative and cannot be met by most rural roadside areas – in other cases, natural dispersion lengths in insufficient. In addition,

these guidelines are based on only two variables and formulated without any empirical data or hydrologic modeling.

This research effort has evaluated the use of natural dispersion at various locations in Washington State. By coupling field experiments with numerical modeling, a relationship between measurable design variables (i.e. saturated hydraulic conductivity, roadway width, rainfall intensity, and roadway embankment (LID) length) has been determined. This relationship is captured by Equation 7 (the LID design equation) and is site specific, allowing highway engineers to tailor natural dispersion requirements to various locations throughout Washington. Equally, the effects of slope on runoff did not correlated through either analysis of field data or numeric modeling. These results indicate that natural dispersion should prove to be a very useful method of stormwater treatment for many major multi-lane highways in eastern Washington as well as most rural arterials throughout the state without the acquisition of additional rights of way. A summary of recommended changes to existing natural dispersion evaluation guidance is as follows:

1) Revises LID slope requirements to include up to 3:1 slopes
2) Incorporate Equation 7 to specify required LID length based on:
    a. Contributory roadway width (ACP)
    b. Measured K_s values
    c. Design rainfall intensities

One remaining challenge in the implementation of the LID design equation is the determination of the saturated hydraulic conductivity related to the LID areas. Current methods of K_s measurement at WSDOT involve the use of soil sieve analyses based on regression data for western Washington soils. While this method is a widely accepted practice for infiltration pond

design, its correlation with surface infiltration values is highly variable and partially dependent on soil sampling procedures and compaction. Similarly, the method currently in use was not intended for surface infiltration facilities. Further study on alternative methods to determine the important K_s parameter, such as direct measurement procedures like the Guelph Permeameter, should therefore be pursued. Regardless, the results of this research clearly produced a reliable method of natural dispersion analysis that should be incorporated into stormwater design procedures in lieu of the current guidelines in Washington State.

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APPENDICES