From:	Garrison, Noah
To:	WB-RB8-SantaAna
Cc:	Kheyfets, Anna; Dyer, Johanna; Devine, Jon; Fischer, Adam@Waterboards
Subject:	Comments on Draft Order R8-2014-0002, North Orange County MS4 Permit
Date:	Friday, June 20, 2014 3:20:46 PM
Attachments:	NRDC North OC Regional MS4 Comment 6-20-14 FINAL.pdf Center for Watershed Protection - Impacts of Impervious Cover on Aquatic Systems.pdf Curriero - Association between Extreme Precipitation and Waterborne Disease Outbreaks 2001.pdf EPA - Reducing Stormwater Costs through LID Strategies and Practices.pdf EPA R3 Capacasa Letter to Sakai re Prince George Cty MS4 Objection 8-8-12.pdf 1991 EPA Elliott Memo.pdf

Dear Mr. Fischer:

Attached please find comments and documents in support from the Natural Resources Defense Council on Draft Order No. R8-2014-0002, the North Orange County MS4 Permit. This email will be followed by 2 additional emails containing documents in support of our comment. Please do not hesitate to contact us with any questions, and we look forward to working with you on the permit adoption process.

Best,
Noah Garrison

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Admitted in California

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Cc:	Kheyfets, Anna; Dyer, Johanna; Devine, Jon; Fischer, Adam@Waterboards
Subject:	Comments on Draft Order R8-2014-0002, North Orange County MS4 Permit - supplemental documents
Date:	Friday, June 20, 2014 3:23:32 PM
Attachments:	Haile 1999 SMBRP EPI Journal Article.pdf Horner - Investigation of Feasibility and Benefits of LID for Ventura County.pdf Horner Gretz - 12-2011 Investigation of Feasibility and Benefits of LID.pdf GAO - Water Quality-Better Data and Evaluation of Urban Runoff Programs.pdf Garn USGS - Effects of lawn fertilizer on nutrient concentration in runoff Wisconsin.pdf Given et al - Regional Public Health Cost Estimates.pdf Regional Board Amicus Brief - Malibu.pdf LA County Mun Stormwater BS 080548.pdf Pruss - 1998 Review of epidemiological studies on health effects.pdf

Supplemental Documents 1

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From: Garrison, Noah
To: WB-RB8-SantaAna
Cc: Kheyfets, Anna; Dyer, Johanna; Devine, Jon; Fischer, Adam@Waterboards
Subject: Comments on Draft Order R8-2014-0002, North Orange County MS4 Permit - supplemental documents
Date: Friday, June 20, 2014 3:25:02 PM
Attachments: Ventura County Low Impact Development Technical Guidance Manual -07-13-2011.pdf

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Supplemental Documents 2

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June 20, 2014

Via electronic mail

Mr. Kurt V. Berchtold
Executive Officer and Members of the Board
California Regional Water Quality Control Board, Santa Ana Region
373 Main Street, Suite 500
Riverside, CA 92501-3348
Email: santaana@waterboards.ca.gov

Re: Comments on Tentative Order R8-2014-0002, North Orange County MS4 Permit

Dear Mr. Berchtold:

On behalf of the Natural Resources Defense Council ("NRDC"), we are writing with regard to the Draft National Pollutant Discharge Elimination System (NPDES) Permit and Waste Discharge Requirements for Orange County Flood Control District, the County of Orange and the Incorporated Cities therein within the Santa Ana Region (Area-wide Urban Runoff from Municipal Separate Storm Sewer Systems ("MS4s")) Draft Permit ***R8-2014-0002***, NPDES Permit No. CAS 618030 ("Draft Permit"). We appreciate the opportunity to submit these comments to the Santa Ana Regional Water Quality Control Board ("Regional Board").

I. Stormwater Runoff is a Leading Source of Water Pollution in the Orange County Region

The U.S. Environmental Protection Agency ("EPA") considers urban runoff to be "one of the most significant reasons that water quality standards are not being met nationwide."[1] As the EPA has stated:

> Most stormwater runoff is the result of the man-made hydrologic modifications that normally accompany development. The addition of impervious surfaces, soil compaction, and tree and vegetation removal result in alterations to the movement of water through the environment. As interception, evapotranspiration, and infiltration are reduced and precipitation is converted to overland flow, these modifications affect not only the characteristics of the developed site but also the

[1] U.S. General Accounting Office (June 2001) *Water Quality: Urban Runoff Programs*, Report No. GAO-01-679.

> watershed in which the development is located. Stormwater has been identified as one of the leading sources of pollution for all waterbody types in the United States. Furthermore, the impacts of stormwater pollution are not static; they usually increase with more development and urbanization.[2]

A 2012 study of the effects of urban development on stream ecosystems by the U.S. Geological Survey showed that urban development impacts stream chemistry, hydrology, habitat, and species composition, and that communities of invertebrate species "Begin to Degrade at the Earliest Stages of Urban Development."[3]

In the North Orange County Region, the Regional Board has found that:

- "The discharge of pollutants from MS4s may cause or threaten to cause the concentrations of pollutants in receiving waters to exceed applicable water quality objectives. Discharges from MS4s may result in alterations to the hydrology of receiving waters that negatively impact their physical integrity. These conditions may impair or threaten to impair designated beneficial uses resulting in a condition of pollution, contamination or nuisance." (Draft Permit, at Finding 11);

- "Land development has created, and continues to create, new sources of non-storm water discharges and pollutants in storm water discharges as human population density increases. This brings higher levels of car emissions, car maintenance wastes, municipal sewage, pesticides, household hazardous wastes, pet wastes, and trash. Development typically converts natural ground cover to impervious surfaces such as paved highways, streets, rooftops, and parking lots. Pollutants deposited on these sources are dumped or washed off by non-storm water or storm water flows into and from the MS4s. As a result of the increased imperviousness in urban areas, less rain water can infiltrate through and flow over vegetated soil where physical, chemical, and biological processes can remove pollutants. Therefore, runoff leaving a developed area can contain greater pollutant loads and have significantly greater runoff volume, velocity, and peak flow rate than pre-development runoff conditions from the same area. Certain best management practices can minimize these impacts to water quality." (Draft Permit, at Finding 12);

- "[C]ommon pollutants in urban runoff include total suspended solids, sediment, pathogens (e.g., bacteria, viruses, protozoa), heavy metals (e.g., cadmium, copper, lead, and zinc), petroleum products and polynuclear aromatic hydrocarbons, synthetic organics (e.g., pesticides, herbicides, and PCBs), nutrients (e.g., nitrogen and phosphorus),

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[2]U.S. Environmental Protection Agency (December 2007) *Reducing Stormwater Costs through Low Impact Development (LID) Strategies and Practices*, at v.
[3]U.S. Geological Survey (2012) *Effects of Urban Development on Stream Ecosystems in Nine Metropolitan Study Areas Across the United States*, at 4; see generally, 1-5. Available at: http://pubs.usgs.gov/circ/1373/. Available at http://pubs.usgs.gov/circ/1373/.

oxygen-demanding substances (e.g., decaying vegetation, animal waste), detergents, and trash." (Draft Permit, at Finding 14); and,

- "Pollutants in runoff discharged from the MS4s risk adversely affecting human health and aquatic organisms. Adverse human health effects include gastrointestinal diseases and infections. Adverse physiological responses to pollutants in runoff include impaired reproduction, growth anomalies and mortality in aquatic organisms. These responses may be the result of different mechanism, including bioaccumulation of toxicants. During bioaccumulation, toxicants carry up the food chain and may affect both aquatic and non-aquatic organism, including human health. Increased volume, velocity, rate, and duration of storm water runoff greatly accelerate the erosion of downstream natural channels. This alters stream channels and habitats and can adversely affect aquatic and terrestrial organisms." (Draft Permit, at Finding 15.)

Discharges of polluted urban runoff result in elevated bacteria levels and increased illness rates among swimmers, and the association between heavy precipitation (leading to increased runoff) and waterborne disease outbreaks is well documented.[4] Swimming or contact with waters contaminated by stormwater runoff can lead to fever, chills, ear infections and discharge, coughing and respiratory ailments, vomiting, diarrhea and other gastrointestinal illness, and skin rashes.[5] In a peer-reviewed evaluation of 22 selected epidemiological studies from around the world, scientists found that 19 of 22 studies showed that adverse health effects were significantly related to fecal indicator bacteria or bacterial pathogens.[6]

The Regional Board itself has acknowledged that "microbial contamination of the beaches from urban runoff and other sources has resulted in a number of health advisories issued by the Orange County Health Officer." (2009 Permit (as amended by Order R8-2010-0062, at Finding 36).) And the health impacts do come at tremendous public health and financial cost—one study demonstrated that swimming at polluted beaches in Orange County caused between 200,000 and 486,200 excess cases of gastroenteritis per year, in turn resulting in annual health costs of between $6.6 and $16.2 million (depending on the epidemiological model used) per year.[7]

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[4] Curriero et al., (August 2001) *The Association Between Extreme Precipitation and Waterborne Disease Outbreaks in the United States, 1949-1994,* American Journal of Public Health, 91:8 1194-1199.

[5] See, e.g., Haile, et al. (1999) *The Health Effects of Swimming in Ocean Water Contaminated by Storm Drain Runoff*, Epidemiology 10(4): 355-63; Haile, R. W. et al (1996) *An Epidemiological Study of Possible Adverse Health Effects of Swimming in Santa Monica Bay,* Santa Monica Bay Restoration Project, 70 pp.

[6] Pruss, A. (1998) *Review of epidemiological studies on health effects from exposure to recreational waters*, International Journal of Epidemiology 27:1-9.

[7] Given, S., et al. (2006) Regional Public Health Cost Estimates of Contaminated Coastal Waters: A Case Study of Gastroenteritis at Southern California Beaches, Environmental Science & Technology 40(16): 4851-4858, at 4856.

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Without question, swimming in stormwater runoff-contaminated water has a high cost for the region. The Draft Permit establishes requirements critical to addressing this pollution.

II. Legal Background

In order to "restore and maintain the chemical, physical, and biological integrity of the Nation's waters," (33 U.S.C. § 1251(a)), the federal Clean Water Act ("CWA") prohibits the discharge of any pollutant from a point source into a water of the United States except as in compliance with the Act. (33 U.S.C. §§ 1311(a), 1342.) Point sources, such as MS4s, can comply with the CWA by obtaining a discharge permit under the National Pollutant Discharge Elimination System ("NPDES") program. (33 U.S.C. § 1342(b), (p).) Regulations under 40 C.F.R. section 122.4(d) prohibit the issuance of a NPDES Permit "[w]hen the imposition of conditions cannot ensure compliance with the applicable water quality requirements of all affected States." Further, renewal permits—like the 2012 Permit at issue—may not contain weaker standards than those contained in the previous permit, except under limited circumstances. (33 U.S.C. § 1342(o); 40 C.F.R. § 122.44(l).) Federal and state laws additionally require implementation of an antidegradation policy that mandates that existing water quality in navigable waters be maintained unless degradation is justified by specific findings. (See, 40 C.F.R. § 131.12(a)(1).)

The CWA requires each state to adopt water quality standards for all waters within its boundaries and submit them to the EPA for approval. (33 U.S.C. §§ 1311(b)(1)(C), 1313.) Water quality standards include maximum permissible pollutant levels that must be sufficiently stringent to protect public health and enhance water quality, consistent with the uses for which the water bodies have been designated. (33 U.S.C. § 1313(c)(2)(A).) They provide the reference point "to prevent water quality from falling below acceptable levels." (*PUD No. 1 of Jefferson County v. Washington Dep't of Ecology* (1994) 511 U.S. 700, 704 [quotation omitted].) States also must identify as impaired any water bodies that fail to meet water quality standards. (33 U.S.C. § 1313(d).)

A. Clean Water Act Section 402(p)

Like all NPDES permits, MS4 permits must ensure that discharges from storm sewers do not cause or contribute to a violation of water quality standards. (33 U.S.C. § 1311(a); 1313; 1341(a); 1342(p).)[8] In addition, for MS4s covered under the NPDES program, permits:

> shall require controls to reduce the discharge of pollutants to the maximum extent practicable, including management practices, control techniques and system, design and engineering methods, and such other provisions as the Administrator or the State determines appropriate for the control of such pollutants.

[8]See, e.g., State Board Order No. WQ 99-05, *Own Motion to Review the Petition of Environmental Health Coalition to Review Waste Discharge Requirements Order No. 96-03*.

(33 U.S.C. § 1342(p)(3)(B)(iii).) The Clean Water Act's "maximum extent practicable" ("MEP") standard does not grant unbridled leeway to Permittees in developing controls to reduce the discharge of pollution. (See, e.g., *Defenders of Wildlife v. Babbitt* (D.D.C. 2001) 130 F. Supp. 2d 121, 131; *Environmental Defense Center, Inc. v. U.S. E.P.A* (9th Cir. 2003) 344 F.3d 832, 853.) The MEP standard "imposes a clear duty on the agency to fulfill the statutory command to the extent that it is feasible or possible." (*Defenders of Wildlife v. Babbitt,* 130 F. Supp. 2d at 131; *Friends of Boundary Waters Wilderness v. Thomas,* 53 F.3d 881, 885 (8th Cir. 1995) ("feasible" means "physically possible"). As one state hearing board held:

> [MEP] means to the fullest degree technologically feasible for the protection of water quality, except where costs are wholly disproportionate to the potential benefits…. This standard requires more of Permittees than mere compliance with water quality standards or numeric effluent limitations designed to meet such standards…. The term "maximum extent practicable" in the stormwater context implies that the mitigation measures in a stormwater permit must be more than simply adopting standard practices. This definition applies particularly in areas where standard practices are already failing to protect water quality….

(*North Carolina Wildlife Fed. Central Piedmont Group of the NC Sierra Club v. N.C. Division of Water Quality* (N.C.O.A.H. October 13, 2006) 2006 WL 3890348, Conclusions of Law 21-22 (internal citations omitted).)

Nor is MEP a static requirement—the standard anticipates and in fact requires new and additional controls to be included with each successive permit. As EPA has explained, NPDES permits, including the MEP standard, will "evolve and mature over time" and must be flexible "to reflect changing conditions." (55 Fed. Reg. 47990, 48052.) "EPA envisions application of the MEP standard as an iterative process. MEP should continually adapt to current conditions and BMP effectiveness and should strive to attain water quality standards. Successive iterations of the mix of BMPs and measurable goals will be driven by the objective of assuring maintenance of water quality standards." (64 Fed. Reg. 68722, 68754.) In other words, successive iterations of permits for a given jurisdiction will necessarily evolve, and contain new, and more stringent requirements for controlling the discharge of pollutants in runoff.

Although requiring compliance with MEP may be sufficient to achieve water quality standards and other common permit terms, the Clean Water Act independently requires that MS4 permits achieve water quality standard compliance.[9] EPA has stated "all permits for MS4s must include

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[9]See, 33 U.S.C. § 1311(a); 1313; 1341(a); 1342(p); 40 C.F.R. § 122.44(d)(1) (permits must contain, as applicable, any requirements necessary to "[a]chieve water quality standards established under section 303 of the CWA, including State narrative criteria for water quality"); Memorandum from E. Donald Elliott, Assistant Administrator and General Counsel, U.S. Environmental Protection Agency, to Nancy J. Marvel, Regional Counsel Region IX, re: Compliance with Water Quality Standards in NPDES Permits Issued to Municipal Separate Storm Sewer Systems, Jan. 9, 1991 ("EPA Elliott Memo"). But see, *Defenders of Wildlife v.*

any requirements necessary to achieve compliance with [water quality standards]."[10] Notwithstanding this requirement, permits also require "such other provisions as the Administrator or the State determines appropriate for the control of such pollutants." This language in section 1342(p) has been held by California courts to grant "the EPA (and/or a state approved to issue the NPDES permit) . . . the discretion to impose 'appropriate' water pollution controls in addition to those that come within the definition of 'maximum extent practicable.'" (*Building Industry Ass'n of San Diego County v. State Water Resources Control Bd.* (2004) 124 Cal.App.4th 866, 883 (citing *Defenders of Wildlife v. Browner* (1999) 191 F.3d 1159, at 1165–1167).) As a result, the MEP standard represents a statutory floor, rather than limit, for permit requirements.[11]

B. Orange County MS4 Permits and State Board Order 99-05

In 2009, the Santa Ana Regional Board adopted an NPDES permit for MS4s in North Orange County, which was intended to address the harm caused by pollutants conveyed via storm drains to surface waters in the North Orange County area. The permit regulated the County of Orange, Orange County Flood Control District, and 26 incorporated cities of Orange County within the Santa Ana Region.

Importantly, the 2009 Permit, as did the previous 2002 permit, contained Receiving Water Limitations ("RWLs"), which required that "discharges from the MS4s shall not cause or contribute to exceedances of receiving water quality standards (designated beneficial uses and water quality objectives) for surface waters or groundwaters." (2009 Permit, at Part VI.1.) The Permittees were directed to begin remedial measures immediately if discharges violate water quality standards. (*Id*., at Part VI.) If exceedances of water quality standards persisted, notwithstanding control measures, the Dischargers were required to "achieve compliance" by preparing a compliance report that identifies the violations and by adopting pollution control measures to correct them. (*Id*.)

Complying with this "iterative process" assisted Dischargers in meeting water quality goals, but did not excuse violations of water quality standards. A long history of MS4 permitting in California confirms this. For example, an earlier MS4 permit for Orange County had included language stating, "The permittees will not be in violation of [the receiving water limitations] so long as they are in compliance with the requirements [of the iterative process set forth in the permit]."[12] Similarly, a permit for Los Angeles County, approved by the State Water Resources

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Browner (9th Cir. 1999) 191 F.3d 1159, 1166 (holding that permitting authority is not required to impose strict water quality-based effluent limitations, but has the authority to do so).

[10] EPA Elliott Memo, at 1; *In re: Government of the District of Columbia Municipal Separate Storm Sewer System* (EPA 2002) 10 E.A.D. 323, 2002 WL 257698.

[11] See also, *Building Industry Ass'n of San Diego County v. State Water Resources Control Bd.* (2004) 124 Cal.App.4th 866, 883; *Defenders of Wildlife v. Browner* (9th Cir. 1999) 191 F.3d 1159, at 1165–1167.

[12] See Order No. 96-31, at Part IV.1.

Control Board ("State Board"), had included language stating "the permittees will not be in violation of [receiving water limitations] so long as they are in compliance with [the iterative process set forth in the permit]."[13] But EPA objected to that provision, (which MS4 permits for Vallejo and Riverside County had additionally adopted), as a "safe harbor," meaning the provision deemed the permittees in compliance with the permit regardless of whether water quality standards were then met. In response, the State Board adopted Order No. 99-05, which directed the Regional Boards to include receiving water limitations language devised by EPA, without a safe harbor provision, into all future MS4 permits.[14] As the Los Angeles Regional Board has rightly pointed out with regard to provisions which excuse compliance with water quality standards, under this framework, "The Regional Board did not include a safe harbor in [its MS4] Permit and, under California law, could not have done so."[15]

III. Permit Provisions

A. The Approach Taken in the Draft Permit Creates Illegal Safe Harbors that Violate Federal Anti-Backsliding and Antidegradation Requirements

Unlike the prior 2009 Permit, which simply states that "discharges from the MS4s shall not cause or contribute to exceedances of receiving water quality standards (designated beneficial uses and water quality objectives) for surface waters or groundwaters," the draft 2014 Permit states that "discharges from the Co-permittees' MS4s must not cause or contribute to exceedances of receiving water quality standards (designated beneficial uses and water quality objectives) for surface or ground waters or cause or contribute to a condition of nuisance *unless* a draft plan . . . has been submitted or, if final, is being fully implemented." (2009 Permit, at Part IV.1.; Draft Permit, at Part IV.A. (emphasis added)). These safe harbors, little different from those objected to by EPA more than a decade ago, render the RWLs inoperative and excuse compliance with both narrative and numeric water quality standards; If a Permittee meets the program requirements for submission of a compliance action plan, it is deemed to *legally* comply with the Draft Permit's RWLs, regardless of whether the RWLs are *actually* achieved. The safe harbor

[13]See, State Board Order No. WQ 98-01, *Own Motion to Review the Petition of Environmental Health Coalition to Review Waste Discharge Requirements Order No. 96-03*, at 6-7.
[14]See, State Board WQ Order 99-05.
[15]Brief of Amicus Curiae California Regional Water Quality Control Board, Los Angeles Region, in *Santa Monica Baykeeper v. City of Malibu* No. CV 08-1465-AHM (PLAx) (C.D. Cal.) (filed Feb. 5, 2010), at 8. The Receiving Water Limitations provisions requiring strict compliance with water quality standards were upheld by a California State Court. (*In re L.A. County Mun. Storm Water Permit Litigation.*, No. BS 080548 at 4-7 (L.A. Super. Ct. Mar. 24, 2005)). That court additionally found that the Receiving Water Limitations did not exceed federal requirements as, "the terms of the Permit taken, as a whole, constitute the Los Angeles Regional Board's definition of MEP, including, but not limited to, the challenged [RWL] Permit Provisions." (*Id.* at 7-8).

provisions violate multiple provisions of the CWA and other federal and state regulations, and render the Draft Permit unlawful.[16]

1. The Draft Permit's Safe Harbor Provisions Violate Federal Anti-Backsliding Requirements

The Clean Water Act and federal regulations prohibit backsliding, or weakening of permit terms, from the previous permit. (See, 33 U.S.C. § 1342(o)(1); 40 C.F.R. § 122.44(l)(1).) By providing a safe harbor waiving requirements to meet Water Quality Standards, the Draft Permit flatly violates these federal requirements.

Courts have found that, for RWL language nearly identical to that of the 2009 Permit (or 2002 Permit), the prohibition against discharges that cause or contribute to a violation of water quality standards requires strict compliance with those standards. "Succinctly put, the [Receiving Water Limitations] incorporate[] the pollution standards promulgated in other agency documents such as the Basin Plan, and prohibit[] stormwater discharges that 'cause or contribute to the violation' of those incorporated standards." (*Natural Resources Defense Council v. Los Angeles County* (9th Cir. 2013) 725 F.3d 1194, 1199.) In contrast, the Draft Permit deems a Permittee submitting a plan for actions to be taken to achieve compliance to be in compliance with RWLs, even if a Permittee's discharges actually cause or contribute to an exceedance of water quality standards. Thus, the Draft Permit excuses discharges of pollution and violations of water quality standards that the previous permit prohibited.

Section 402(o) of the Clean Water Act (33 U.S.C. § 1342(o)), generally prohibits relaxation of, among other things, an effluent limitation[17] "necessary to meet water quality standards . . . schedules of compliance, established pursuant to any State law or regulations . . . or any other Federal law or regulation, or required to implement any applicable water quality standard established pursuant to" the CWA. (See, 33 U.S.C. § 1342(o)(1) (referencing 33 U.S.C. § 1311(b)(1)(C).)[18] The safe harbors, which violate this prohibition against backsliding, fail to

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[16] These exemptions from requirements to meet the RWLs are also imprudent; water quality standards are established at levels necessary to protect the environment and public health. Failing to ensure compliance with water quality standards, through provisions that deem Permittees to be in compliance regardless of whether water quality standards are actually met, does *not* protect the environment, and does *not* protect public health.

[17] Receiving Water Limitations constitute effluent limitations under the CWA. (See 33 U.S.C. § 1362(11).) But even if this were not the case, the safe harbors would still be unlawful, as EPA's anti-backsliding regulations under 40 C.F.R. § 122.44(l)(1) require that "effluent limitations, *standards or conditions* must be at least as stringent as the final effluent limitations, standards, or conditions in the previous permit. . . ." (Emphasis added.)

[18] EPA has recognized that even providing additional time for compliance for a provision required by the previous permit violates anti-backsliding requirements. (Letter from Jon M. Capacasa, Director Water Protection Division, EPA Region III to Jay Sakai, Maryland Department of the Environment, re: Specific Objection to Prince George's County Phase I Municipal Separate

satisfy any enumerated exception to the provision. (See, 33 U.S.C. § 1313(d)(4); section 402(o)(2).)[19] Neither are they lawful under section 402(o)(3), which serves as a "*safety clause* that provides an absolute limitation on backsliding,"[20] and states that in no event shall a permit "be renewed, reissued, or modified to contain a less stringent effluent limitation if the implementation of such limitation would result in a violation of a water quality standard" under 33 U.S.C. § 1313. (33 U.S.C. § 1342(o)(3).) The Draft Permit, by explicitly excusing violations of RWLs which prohibit discharges that cause or contribute to a violation of water quality standards, fails to meet this federally mandated minimum level of protection.

2. The Draft Permit's Safe Harbor Provisions Violate State and Federal Antidegradation Requirements

The overall goal of the Clean Water Act is the complete elimination of the discharge of pollutants into waters of the United States. (33 U.S.C. § 1251(a)(1).) To help meet this goal, states must implement an antidegradation policy. However, the permit does not comply with applicable antidegradation requirements.

The federal antidegradation policy contains a three "Tier" test for determining when increases in pollutant loadings or adverse changes to water quality may be allowed.[21] (40 C.F.R. § 131.12.) Tier I antidegradation analysis applies to *all* waters of the United States,[22] applying "a minimum level of protection to all waters . . . even seriously degraded water bodies . . . prohibiting any additional pollution that would affect existing uses."[23]

NPDES permit renewals or modifications such as the Draft Permit are subject to both state and federal antidegradation requirements, which mandate that existing water quality in navigable waters be maintained, unless degradation is justified based on specific findings.[24] In no case

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Storm Sewer System (MS4) Permit MD0068284, at 3. The additional time allotted by the new Permit to achieve compliance with RWLs, required in the 2001 Permit, for Permittees developing a WMP or an EWMP therefore constitutes a less stringent limitation.

[19]See also, EPA (September 2010) NPDES Permit Writers' Manual ("NPDES Manual), at 7-1 to 7-3.

[20]See EPA, NPDES Manual at 7-4.

[21]California has established a state antidegradation policy, which incorporates the federal antidegradation policy and establishes additional requirements. (See, State Board Resolution 68-16; *see also In the Matter of the Petition of Rimmon C. Fay*, State Board Order No. WQ 86-17 at 16-19.)

[22]64 Fed. Reg. 46058, 46063, *Revisions to the National Pollutant Discharge Elimination System Program and Federal Antidegradation Policy in Support of Revisions to the Water Quality Planning and Management Regulation*.

[23]Brawer, J.M., "Antidegradation Policy and Outstanding Natural Resource Waters in the Northern Rocky Mountain States," 20 Pub. Land & Resources L. Rev. 13, 18 (1999).

[24]*See*, SWRCB Order No. WQ 86-17; EPA, Region IX, *Guidance on Implementing the Antidegradation Provisions of 40 C.F.R. § 131.12,* at 2-4 (June 3, 1987) ("EPA Antidegradation Guidance").

may water quality be lowered to a level that would interfere with existing or designated uses. By potentially allowing for discharges from the MS4 to violate water quality standards, in effect degrading those waters, while deeming Permittees to be in compliance with Permit requirements, the Permit fails to properly implement antidegradation requirements. Nor has the Regional Board provided any data, analysis, or findings, which must be accomplished on a pollutant-by-pollutant and beneficial-use-by beneficial use basis, to support degradation. (*See, Associacion de Gente Unida for El Agua v. Central Valley Regional Board* (2012) 210 Cal.App.4th 1255, 1268-69, 1271-72 (citing St. Water Res. Control Bd., Guidance Memorandum (Feb. 16, 1995); 40 CFR 131.12(a)(1).)[25] In past instances when a Regional Board has failed to provide adequate findings to verify that water quality will be maintained, the State Board has remanded the orders to the Regional Board for further proceedings, and the Draft Permit should be revised to avoid that event here.[26]

B. The Draft Permit's Development Planning Requirements Must Require On-Site Retention of at least the 85th Percentile Storm

We strongly support that the Draft Permit establishes requirements for new development and redevelopment projects to retain stormwater runoff on-site. A principal reason to adopt such an approach is the superior pollutant load reduction capacity of low impact development practices that retain runoff on-site, for a variety of climatic scenarios, including for the North Orange County region.[27]

The Draft Permit requires, under one provision, that the runoff from the 85th percentile, 24-hour rain event must be retained on-site. (Draft Permit, at XII.D(1)(a)-(b).) This requirement, which was also included in the previous North Orange County permit, results in retention of stormwater runoff with no off-site discharge in the large majority of storms. The 85th percentile requirement is consistent with on-site retention requirements of other permits throughout California, as well

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[25]The Permit's reference to antidegradation is limited to a cursory summary of the legal requirements, and a conclusion that "[t]his Order requires the Co-permittees to implement programs and policies necessary to improve water quality; the order does not allow any degradation of water quality." (Draft Permit, at p. 13, Finding 25.) Simply claiming that no degradation will occur does not satisfy the requirements of the Clean Water Act. (*Associacion de Gente Unida*, 210 Cal.App.4th at 1260-61; see also, *American Funeral Concepts-American Cremation Soc'y v. Board of Funeral Directors and Embalmers* (1982) 136 Cal.App.3d 303, 309.)
[26]See, e.g., SWRCB Order No. WQ 86-17, at 28.
[27]See, Dr. Richard Horner and Jocelyn Gretz (December 2011) Investigation of the Feasibility and Benefits of Low-Impact Site Design Practices Applied to Meet Various Potential Stormwater Runoff Regulatory Standards ("Horner and Gretz Runoff Study"); see also, Horner, Richard. Report for Ventura County; Horner, Richard. Initial Investigation for San Francisco Bay Area; Horner, Richard. Supplementary Investigation for San Francisco Bay Area; Horner, Richard. Report for San Diego Region.

as in permits and ordinances found in all corners of the United States. Similar or more stringent requirements are included in the following permits:

Ventura County: MS4 permit requires on-site retention of ninety-five percent of rainfall from the 85th percentile storm; off-site mitigation allowed if on-site retention is technically infeasible;[28]

San Diego: MS4 permit requires on-site retention of the 85th percentile storm;[29]

However, the 85th percentile standard is actually less stringent than required by permits in many other parts of the county. For example, permits in the following locations require retention that generally exceeds the 85th percentile storm volume for much of North Orange County:

Washington, D.C.: MS4 permit requires retention of the first 1.2 inches of stormwater (which represents the 90th percentile storm) for all new development and redevelopment over 5,000 square feet.[30]

West Virginia: Statewide Phase II MS4 permit requires on-site retention of "the first one inch of rainfall from a 24-hour storm" event unless infeasible;[31] and,

Philadelphia, PA: Infiltrate the first one inch of rainfall from all impervious surfaces; if on-site infiltration is infeasible, the same performance must be achieved off-site.[32]

Further, research conducted by Dr. Richard Horner, a member of the National Academy of Sciences Panel on Reducing Stormwater Discharge Contributions to Water Pollution demonstrates that, for five different types of land use development or redevelopment projects in Southern California, the full 85th percentile, or even the full 95th percentile, 24-hour precipitation event could be retained on-site using *only* infiltration practices on sites overlying soils classified as Group C (typically containing 20 to 40 percent clay) under the Natural Resources

[28]Los Angeles Regional Water Quality Control Board (July 8, 2010) Ventura County Municipal Separate Stormwater National Pollutant Discharge Elimination System (NPDES) Permit; Order No. R4-2009-0057; NPDES Permit No. CAS004002.
[29]San Diego Regional Water Quality Control Board (December 16, 2014) South Orange County MS4 Permit, Order No. R9-2013-0001, NPDES Permit No. CAS0109266.
[30]U. S. EPA (2011) Fact Sheet, National Pollutant Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System (MS4) Permit No. DC0000221 (Government of the District of Columbia).
[31]State of West Virginia Department of Environmental Protection, Division of Water and Waste Management, General National Pollution Discharge Elimination System Water Pollution Control Permit, NPDES Permit No. WV0116025 at 13-14 (June 22, 2009).
[32]City of Philadelphia (Jan. 29, 2008) Stormwater Management Guidance Manual 2.0, at 1.1, available at.

Conservation Service (NRCS) major soil orders classification scheme.[33] Critically, even for sites overlying Group D soils (typically 40 percent or more clay with substantially restricted water transmissivity) and assuming *no* infiltration was feasible, greater than 50 percent of the 85th percentile storm (or between 37 and 62 percent of annual runoff) could be retained at each development type using only rooftop runoff dispersion or rooftop harvest and reuse techniques.[34] Additional retention under these scenarios could be achieved through use of evaporation practices, green roofs, or, in cases where some infiltration is feasible, use of infiltration BMPs.

NRDC does support use of regional (or "off-site") retention projects that may provide multiple benefits, including increased local water supply, where runoff is *conveyed* from a project site to a regional facility that will retain that runoff, albeit at a different location, with no discharge to receiving waters. This process typically does not implicate significant water quality concerns—where the same, specific quantum of runoff from the project is ultimately retained, 100 percent of the pollution contained in that particular volume of water will be prevented from reaching receiving waters. In contrast, where a project performs off-site mitigation or retrofit at some other location within the same watershed or sub-watershed that is not hydrologically connected to the original project site, it raises substantial concerns as to whether the alternate location will provide equal water quality benefits to the receiving surface water. Among the issues presented by this form of off-site mitigation are: whether the off-site mitigation will be performed at a similar land use type, whether the mitigation project will achieve equivalent pollutant load reduction, and if so, what pollutants it will be monitored for. In practice it may prove exceedingly difficult to assess the equivalency of benefits to surface water quality from retention at one site to the next.

1. The Draft Permit Must Require a Determination that it is Technically Infeasible to Retain the Design Storm On-Site Before Biofiltration is Authorized.

While we support the inclusion of strong retention standards for stormwater runoff, we are concerned by Draft Permit provisions allowing for use of biofiltration and off-site mitigation even where on-site retention is feasible. Because retention of the 85th Percentile Storm event has been established as MEP in California Permits,[35] the project proponent must meet this standard or demonstrate that it cannot be met.

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[33] Horner and Gretz Runoff Study, at Table 16 p. 35; Natural Resources Conservation Service, Distribution Maps of Dominant Soil Orders (http://soils.usda.gov/technical/classification/orders/, last accessed December 16, 2011).
[34] Horner and Gretz Runoff Study, at Table 16 p. 35; 27-34. We note as well that even in areas characterized regionally as underlain by D soils, site specific investigation may establish substantial potential for infiltration of runoff.
[35] See, e.g., Ventura County MS4 Permit, Order No. R4-2009-0057; San Francisco Bay Area MS4 Permit, Order No. R2-2009-0074; North Orange County MS4 Permit, Order No. R8-2009-0030; South Orange County MS4 Permit, Order No. R9-2009-0002.

The jurisdictions identified in sections above have recognized the paramount importance of mandating on-site retention of a certain quantity of stormwater since, in contrast to retention practices, which ensure that 100 percent of the pollutant load in the retained volume of runoff does not reach receiving waters, biofiltration practices (or tree-box filters and other similar practices) that treat and then discharge runoff through an underdrain result in the release of pollutants to receiving waters. Indeed, in order to achieve equivalent pollutant load reduction benefits to the use of on-site retention, biofiltration practices would have to be 100 percent effective at filtering pollutants from the same volume of runoff, which they are invariably not. As a result, while biofiltration practices (or conventional flow-through practices) may be appropriate for on-site treatment when coupled with an off-site mitigation requirement in cases of technical infeasibility (discussed further below), they are not a proper substitute for low impact development ("LID") practices that retain water on-site.

This conclusion is borne out by data presented in the Draft Ventura County Technical Guidance Manual, which estimated pollutant removal efficiency for total suspended solids to be 54-89 percent, and for total zinc to be 48-96 percent.[36] Biofiltration has additionally been shown to be a particularly ineffective method of pollutant removal for addressing nitrogen or phosphorous, two common contaminants found in stormwater.[37] The Draft Ventura Technical Guidance report, for example, indicated that biofiltration achieves pollutant removal efficiency for total nitrogen at between only 21-54 percent,[38] as compared with 100 percent for runoff retained on-site. As a result, even where a multiplier is applied requiring 1.5 times as much runoff be treated using biofiltration as would otherwise be retained, biofiltration may achieve substantially less pollution reduction as would retention.

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[36] Ventura County Low Impact Development Technical Guidance Manual, July 13, 2011, at D-7.

[37] Lawn irrigation has been identified as a "hot spot" for nutrient contamination in urban watersheds—lawns "contribute greater concentrations of Total N, Total P and dissolved phosphorus than other urban source areas . . . source research suggests that nutrient concentrations in lawn runoff can be as much as four times greater than other urban sources such as streets, rooftops or driveways." Center for Watershed Protection (March 2003) *Impacts of Impervious Cover on Aquatic Systems* at 69; see also H.S. Garn (2002) *Effects of lawn fertilizer on nutrient concentration in runoff from lakeshore lawns, Lauderdale Lakes, Wisconsin*. U.S. Geological Survey Water- Resources Investigations Report 02-4130 (In an investigation of runoff from lawns in Wisconsin, runoff from fertilized lawns contained elevated concentrations of phosphorous and dissolved phosphorous).

[38] Ventura County Low Impact Development Technical Guidance Manual, July 13, 2011, at D-7. See also, BASMAA (December 1, 2010) *Draft Model Bioretention Soil Media Specifications-MRP Provision C.3.c.iii*, at Annotated Bibliography section 3.0 (noting nutrient removal from synthetic stormwater runoff demonstrated only 55 to 65 percent of total Kjeldahl nitrogen removal and that only 20 percent of nitrate is removed from the runoff).

IV. Conclusion

We appreciate this opportunity to comment on the Draft Permit. Please feel free to contact us with any questions or concerns you may have.

Sincerely,

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Staff Attorney*
Natural Resources Defense Council
**Admitted in California*

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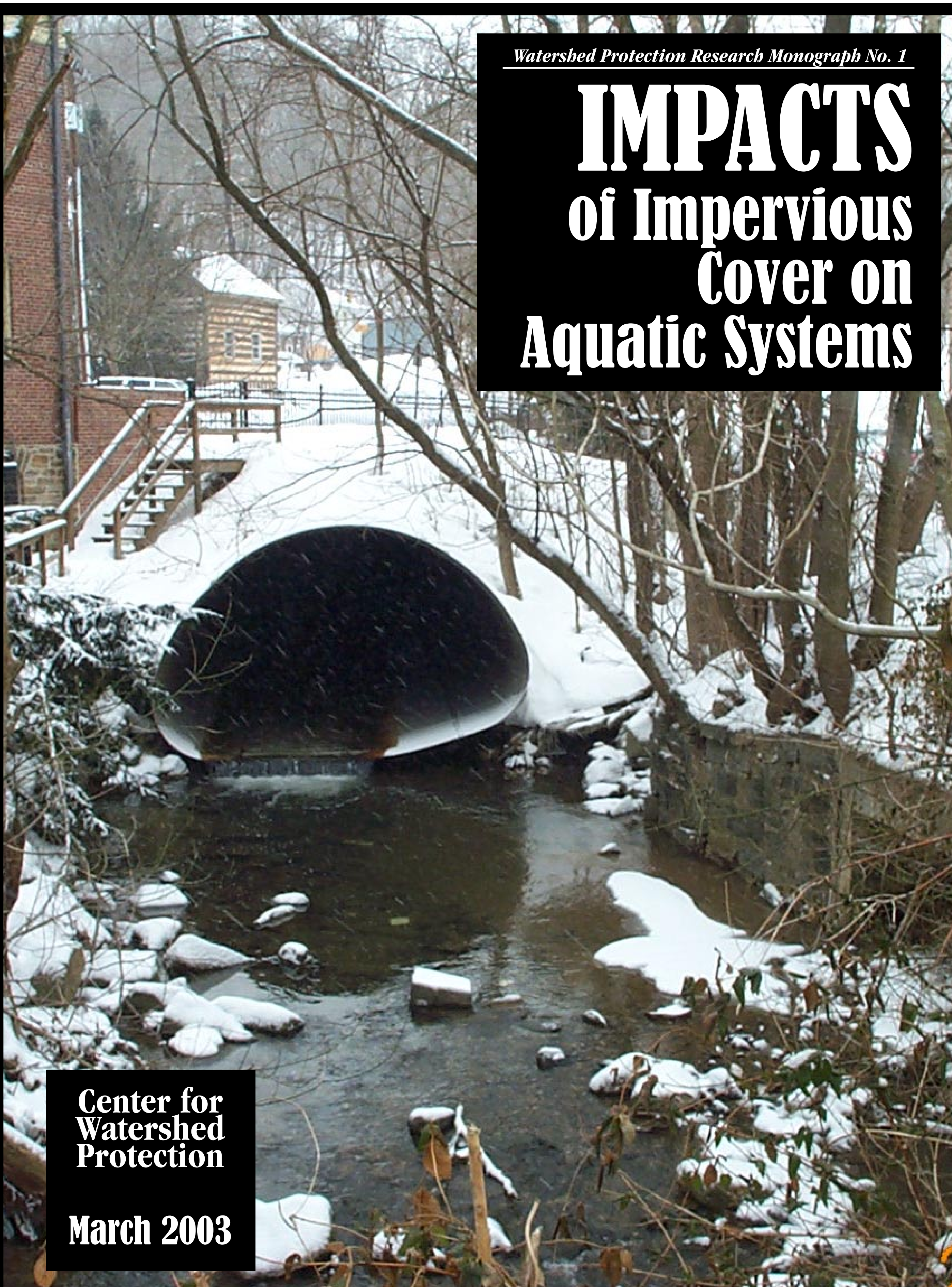
Watershed Protection Research Monograph No. 1

IMPACTS of Impervious Cover on Aquatic Systems



Center for Watershed Protection

March 2003

Cover photograph Ellicott City, Maryland 2003.
Courtesy Anne Kitchell, Center for Watershed Protection.

Watershed Protection Research Monograph No. 1

Impacts of Impervious Cover on Aquatic Systems

March 2003

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Foreword

We are extremely pleased to launch the first edition of a new series called *Watershed Protection Research Monographs*. Each monograph will synthesize emerging research within a major topical area in the practice of watershed protection. The series of periodic monographs will replace our journal *Watershed Protection Techniques,* which lapsed in 2002. We hope this new format will provide watershed managers with the science and perspectives they need to better protect and restore their local watersheds.

This monograph was written to respond to many inquiries from watershed managers and policy makers seeking to understand the scientific basis behind the relationship between impervious cover and the health of aquatic ecosystems. It reviews more than 225 research studies that have explored the impact of impervious cover and other indicators of urbanization on aquatic systems. This report comprehensively reviews the available scientific data on how urbanization influences hydrologic, physical, water quality, and biological indicators of aquatic health, as of late 2002.

Our intention was to organize the available scientific data in a manner that was accessible to watershed leaders, policy-makers and agency staff. In addition, the research itself, which spans dozens of different academic departments and disciplines, was conducted in many different eco-regions, climatic zones, and stream types. In order to communicate across such a wide audience, we have resorted to some simplifications, avoided some important particulars, refrained from some jargon, and tried, wherever possible, to use consistent terminology. Thus, the interpretations and conclusions contained in this document are ours alone, and our readers are encouraged to consult the original sources when in doubt.

We would also like to note that the Center for Watershed Protection and the University of Alabama are currently developing a major national database on stormwater quality. The database will contain nearly 4,000 station-storm events collected by municipalities as part of the U.S. EPA's National Pollutant Discharge Elimination System (NPDES) Phase I Stormwater Permit Program. We anticipate releasing a data report in late 2003 that will provide a much needed update of stormwater event mean concentrations (EMCs).

As of this writing, many research efforts are underway that will further test and refine these relationships (most notably, the U.S. Geological Survey gradients initiative, but also many other local, state and academic efforts). We hope that this report provides a useful summary of the existing science, suggests some directions for new research, and stimulates greater discussion of this important topic in watershed management. We also feel it is time for a major conference or symposium, where this diverse community can join together to discuss methods, findings and the important policy implications of their research.

Acknowledgments

Putting this first research monograph together took a lot of energy, editing and analysis, and many Center staff devoted their time and energy over the last two years to get it done. The project team consisted of Karen Cappiella, Deb Caraco, Samantha Corbin, Heather Holland, Anne Kitchell, Stephanie Linebaugh, Paul Sturm, and Chris Swann. Special thanks are extended to Tiffany Wright, who worked tirelessly to assemble, edit and otherwise polish the final draft.

I am also grateful to Michael Paul of Tetratech, Inc., who graciously provided us with an extensive literature review from his PhD days at the University of Georgia that contained many obscure and hard to find citations. Portions of this monograph were developed as part of a literature review conducted as part of a work assignment for the U.S. EPA Office of Wastewater Management in 2001, which proved indispensable in our efforts. Lastly, I would like to thank the hundreds of scientists who have contributed their time and data to explore and test the relationships between urbanization and aquatic health.

Tom Schueler
Center for Watershed Protection

Table of Contents

Foreword i
Acknowledgments iii
List of Acronyms and Abbreviations xi

Chapter 1: Introduction
1.1 Is Impervious Cover Still Important? A Review of Recent Stream Research 1
1.1.1 Strength of the Evidence for the ICM 3
1.1.2 Reinterpretation of the ICM 5
1.1.3 Influence of Watershed Treatment Practices on the ICM 9
1.1.4 Recommendations for Further ICM Research 12
1.2 Impacts of Urbanization on Downstream Receiving Waters 14
1.2.1 Relationship Between Impervious Cover and Stormwater Quality 14
1.2.2 Water Quality Response to Stormwater Pollution 15
1.2.3 Effect of Watershed Treatment on Stormwater Quality 18
1.3 Implications of the ICM for Watershed Managers 21
1.3.1 Management of Non-Supporting Streams 21
1.3.2 Use of the ICM for Urban Stream Classification 22
1.3.3 Role of the ICM In Small Watershed Planning 22
1.4 Summary 24

Chapter 2: Hydrologic Impacts of Impervious Cover
2.1 Introduction 25
2.2 Increased Runoff Volume 27
2.3 Increased Peak Discharge Rate 30
2.4 Increased Bankfull Flow 31
2.5 Decreased Baseflow 34
2.6 Conclusions 37

Chapter 3: Physical Impacts of Impervious Cover
3.1 Difficulty in Measuring Habitat 40
3.1.1 The Habitat Problem 40
3.2 Changes in Stream Geometry 42
3.2.1 Channel Enlargement 42
3.2.2 Effect of Channel Enlargement on Sediment Yield 45
3.3 Effect on Composite Measures of Stream Habitat 46
3.4 Effect on Individual Elements of Stream Habitat 47
3.4.1 Bank Erosion and Bank Stability 47
3.4.2 Embeddedness 47
3.4.3 Large Woody Debris (LWD) 49
3.4.4 Changes in Other Individual Stream Parameters 49
3.5 Increased Stream Warming 50
3.6 Alteration of Stream Channel Networks 52
3.6.1 Channel Modification 52
3.6.2 Barriers to Fish Migration 53
3.7 Conclusion 54

Chapter 4: Water Quality Impacts of Impervious Cover
4.1 Introduction 55
4.2 Summary of National and Regional Stormwater Pollutant Concentration Data 56
4.2.1 National Data 56
4.2.2 Regional Differences Due to Rainfall 56
4.2.3 Cold Region Snowmelt Data 58
4.3 Relationship Between Pollutant Loads and Impervious Cover: The Simple Method 61
4.4 Sediment 63

4.4.1 Concentrations 63
4.4.2 Impacts of Sediment on Streams 63
4.4.3 Sources and Source Areas of Sediment 64
4.5 Nutrients 67
4.5.1 Concentrations 67
4.5.2 Impacts of Nutrients on Streams 68
4.5.3 Sources and Source Areas of Nutrients 69
4.6 Trace Metals 71
4.6.1 Concentrations 71
4.6.2 Impacts of Metals on Streams 72
4.6.3 Sources and Source Areas of Trace Metals 73
4.7 Hydrocarbons: PAH, Oil and Grease 75
4.7.1 Concentrations 75
4.7.2 Impacts of Hydrocarbons on Streams 75
4.7.3 Sources and Source Areas of Hydrocarbons 76
4.8 Bacteria & Pathogens 77
4.8.1 Concentrations 77
4.8.2 Impacts of Bacteria and Pathogens on Streams 79
4.8.3 Sources and Source Areas of Bacteria and Pathogens 80
4.9 Organic Carbon 82
4.9.1 Concentrations 82
4.9.2 Impacts of Organic Carbon on Streams 82
4.9.3 Sources and Source Areas of Total Organic Carbon 82
4.10 MTBE 83
4.10.1 Concentrations 83
4.10.2 Impacts of MTBE on Streams 83
4.10.3 Sources and Source Areas of MTBE 84
4.11 Pesticides 85
4.11.1 Concentrations 86
4.11.2 Impacts of Pesticides on Streams 86
4.11.3 Sources and Source Areas of Pesticides 87
4.12 Deicers 88
4.12.1 Concentrations 89
4.12.2 Impacts of Deicers on Streams 89
4.12.3 Sources and Source Areas of Deicers 90
4.13 Conclusion 91

Chapter 5: Biological Impacts of Impervious Cover
5.1 Introduction 93
5.2 Indicators and General Trends 95
5.2.1 Biological Indicators 95
5.2.2 Watershed Development Indices 95
5.2.3 General Trends 97
5.3 Effects on Aquatic Insect Diversity 100
5.3.1 Findings Based on Impervious Cover Indicators 100
5.3.2 Findings Based on Other Development Indicators 104
5.4 Effects on Fish Diversity 105
5.4.1 Findings Based on Impervious Cover Indicators 105
5.4.2 Findings Based on other Development Indicators 110
5.5 Effects on Amphibian Diversity 112
5.6 Effects on Wetland Diversity 114
5.7 Effects on Freshwater Mussel Diversity 115
5.8 Conclusion 116

References 117
Glossary 137

List of Tables

1 The Strength of Evidence: A Review of Current Research on Urban Stream Quality Indicators 4
2 Land Use/IC Relationships for Suburban Areas of the Chesapeake Bay 9
3 Summary of Urban Stormwater Pollutant Loads on Quality of Receiving Waters 14
4 The Effectiveness of Stormwater Treatment Practices in Removing Pollutants - Percent Removal Rate 18
5 Median Effluent Concentrations from Stormwater Treatment Practices 19
6 Additional Considerations for Urban Stream Classification 23
7 Research Review of Increased Runoff Volume and Peak Discharge in Urban Streams 28
8 Hydrologic Differences Between a Parking Lot and a Meadow 29
9 Comparison of Bulk Density for Undisturbed Soils and Common Urban Conditions 29
10 Research Review of Increased Bankfull Discharge in Urban Streams 32
11 Research Review of Decreased Baseflow in Urban Streams 34
12 Physical Impacts of Urbanization on Streams 41
13 Research Review of Channel Enlargement and Sediment Transport in Urban Streams 43
14 Research Review of Changes in Urban Stream Habitat 48
15 Research Review of Thermal Impacts in Urban Streams 50
16 National EMCs for Stormwater Pollutants 57
17 Regional Groupings by Annual Rainfall Amount 58
18 Stormwater Pollutant EMCs for Different U.S. Regions 59
19 Mean and Median Nutrient and Sediment Stormwater Concentrations for Residential Land Use Based on Rainfall Regions 59
20 EPA 1986 Water Quality Standards and Percentage of Metal Concentrations Exceeding Water Quality Standards by Rainfall Region 60
21 Runoff and Pollutant Characteristics of Snowmelt Stages 60
22 EMCs for Total Suspended Solids and Turbidity 63
23 Summary of Impacts of Suspended Sediment on the Aquatic Environment 64
24 Summary of Impacts of Deposited Sediments on the Aquatic Environment 64
25 Sources and Loading of Suspended Solids Sediment in Urban Areas 65
26 Source Area Geometric Mean Concentrations for Suspended Solids in Urban Areas 66
27 Mean TSS Inflow and Outflow at Uncontrolled, Controlled and Model Construction Sites 66
28 EMCs of Phosphorus and Nitrogen Urban Stormwater Pollutants 67
29 Source Area Monitoring Data for Total Nitrogen and Total Phosphorus in Urban Areas 69
30 EMCs and Detection Frequency for Metals in Urban Stormwater 71
31 Average Total Recoverable and Dissolved Metals for 13 Stormwater Flows and Nine Baseflow Samples from Lincoln Creek in 1994 72
32 Percentage of In-situ Flow-through Toxicity Tests Using Daphnia *magna* and Pimephales *promelas* with Significant Toxic Effects from Lincoln Creek 73
33 Metal Sources and Source Area "Hotspots" in Urban Areas 74
34 Metal Source Area Concentrations in the Urban Landscape 74
35 Hydrocarbon EMCs in Urban Areas 75
36 Bacteria EMCs in Urban Areas 78
37 *Cryptosporidium* and *Giardia* EMCs 79
38 Percent Detection of Giardia cysts and Cryptosporidium oocysts in Subwatersheds and Wastewater Treatment Plant Effluent in the New York City Water Supply Watersheds 80
39 Typical Coliform Standards for Different Water Uses 80
40 EMCs for Organic Carbon in Urban Areas 82
41 MTBE Detection Frequency 83
42 Median Concentrations and Detection Frequency of Herbicides and Insecticides in Urban Streams 85
43 Use and Water Quality Effect of Snowmelt Deicers 88
44 EMCs for Chloride in Snowmelt and Stormwater Runoff in Urban Areas 89
45 Summary of State Standards for Salinity of Receiving Waters 90

46 Review of Stressors to Urban Streams and Effects on Aquatic Life 94
47 Examples of Biodiversity Metrics Used to Assess Aquatic Communities 96
48 Alternate Land Use Indicators and Significant Impact Levels 98
49 Recent Research Examining the Relationship Between IC and Aquatic Insect Diversity in Streams 101
50 Recent Research Examining the Relationship of Other Indices of Watershed Development on Aquatic Insect Diversity in Streams 102
51 Recent Research Examining the Relationship Between Watershed IC and the Fish Community 106
52 Recent Research Examining Urbanization and Freshwater Fish Community Indicators 108
53 Recent Research on the Relationship Between Percent Watershed Urbanization and the Amphibian Community 113
54 Recent Research Examining the Relationship Between Watershed Development and Urban Wetlands 114

List of Figures

1 Impervious Cover Model 2
2 Typical Scatter Found in IC/Stream Quality Indicator Research 5
3 Relationship of IC and FC in Puget Sound Subwatersheds 6
4 The Double Scatter Problem: Difficulties in Detecting the Effect of Watershed Treatment .. 10
5 Estimated Phosphorous Load as a Function of IC, Discounted Stormwater Treatment and Better Site Design 19
6 Altered Hydrograph in Response to Urbanization 25
7 Runoff Coefficient vs. IC 27
8 Discharge for Urban and Rural Streams in North Carolina 30
9 Effect on Flood Magnitudes of 30% Basin IC 31
10 Relationship of Urban/Rural 100-year Peak Flow Ratio to Basin Development Factor and IC 31
11 Increase in Bankfull Flows Due to Urbanization 32
12 Increase in Number of Exceedences of Bankfull Flow Over Time With Urbanization 33
13 Percent of Gage Reading Above Mean Annual Flow 33
14 Relationship Between Baseflow and Watershed Impervious Cover 34
15 Baseflow Response to Urbanization 35
16 Relationship Between Percentage Baseflow and Percent IC 35
17 Effect of IC on Summer Baseflow (Corrected for Catchment Area) 36
18 Effect of Watershed IC on Summer Stream Velocity 36
19 Urban Stream Channels with Progressively Greater IC 40
20 Increased Shear Stress from a Hydrograph 42
21 Stream Channel Enlargement in Watts Branch 44
22 Ultimate Channel Enlargement 45
23 Relationship Between Habitat Quality and IC in Maine Streams 46
24 Fine Material Sediment Deposition as a Function of IC 47
25 LWD as a Function of IC 49
26 Stream Temperature Increase in Response to Urbanization 51
27 Drainage Network of Rock Creek, D.C. and Four Mile Creek, VA Before and After Urbanization 52
28 Fish Migration Barriers in the Urbanized Anacostia Watershed 53
29 Snowmelt Runoff Hydrograph 60
30 The Simple Method - Basic Equations 61
31 TSS from Bank Erosion vs. IC 65
32 Nitrate-Nitrogen Concentration in Stormwater Runoff 68
33 Total Phosphorus Concentration in Stormwater 68
34 Total Phosphorus From Bank Erosion as a Function of IC 70
35 Fecal Coliform Levels in Urban Stormwater 77

36 Relationship Between IC and Fecal Coliform Concentrations ... 79
37 MTBE Concentrations in Surface Water from Eight Cities ... 83
38 Concentrations of Pesticides in Stormwater in King County, WA ... 87
39 U.S. Highway Salt Usage Data ... 88
40 Combined Fish and Benthic IBI vs. IC ... 98
41 Relationship Between B-IBI, Coho/Cutthroat Ratios, and Watershed IC ... 99
42 Index for Biological Integrity as a Function of Population Density ... 99
43 Trend Line Indicating Decline in Benthic IBI as IC Increases ... 103
44 Compilation of Puget Lowland Watershed Biological Data ... 103
45 IC and IBI at Stream Sites in the Patapsco River Basin, MD ... 103
46 IC vs. Aquatic Insect Sensitivity - EPT scores in Delaware Streams ... 103
47 Average and Spring EPT Index Values vs. % IC in 20 Small Watersheds in Maine ... 104
48 Fish IBI vs. Watershed IC for Streams in Patapsco River Basin, MD ... 105
49 Fish IBI and Number of Species vs. % IC in Wisconsin Streams ... 107
50 IC and Effects on Fish Species Diversity in Four Maryland Subwatersheds ... 107
51 Coho Salmon/Cutthroat Trout Ratio for Puget Sound ... 109
52 Mean Proportion of Fish Taxa in Urban and Non-Urban Streams, Valley Forge Watershed, PA ... 110
53 Relationship Between Watershed Population Density and Stream IBI Scores ... 111
54 Amphibian Species Richness as a Function of Watershed IC in Puget Sound Lowland Wetlands ... 113

Acronyms and Abbreviations

B-IBI	Benthic Index of Biotic Integrity
BOD	Biological Oxygen Demand
BSD	Better Site Design
C-IBI	Combined Index of Biotic Integrity
cfs	cubic feet per second
COD	Chemical Oxygen Demand
CSO	Combined Sewer Overflow
Cu	Copper
DOC	Dissolved Organic Carbon
du/ac	dwelling units per acre
EMC	Event Mean Concentration
EPT	Ephemeroptera, Plecoptera and Trichoptera
FC	Forest Cover
GIS	Geographic Information Systems
IBI	Index of Biotic Integrity
IC	Impervious Cover
ICM	Impervious Cover Model
lbs/ac	pounds per acre
LWD	Large Woody Debris
mg/kg	milligrams per kilogram
mg/l	milligrams per liter (equal to ppm)
MPN	Most Probable Number
MTBE	Methyl Tertiary-Butyl Ether
N	Number of Studies
N/R	data not reported
NO_2	Nitrite
NO_3	Nitrate
NO_x	Nitrogen Oxides
NPDES	National Pollutant Discharge Elimination System
NTU	Nephelometric Turbidity Unit
NURP	National Urban Runoff Program
PAH	Polycylic Aromatic Hydrocarbons
PCB	Polychlorinated Biphenyl
ppb	Parts per billion (equal to ug/l)
ppm	Parts per million (equal to mg/l)
RBP	Rapid Bioassessment Protocol
SLAMM	Source Loading Assessment/ Management Model
SPMD	Semi-Permeable Membrane Device
SSO	Sanitary Sewer Overflow
STP	Stormwater Treatment Practice
TC	Turf Cover
TDS	Total Dissolved Solids
TKN	Total Kjeldhal Nitrogen
TMDL	Total Maximum Daily Load
Total N	Total Nitrogen
Total P	Total Phosphorous
TOC	Total Organic Carbon
TSS	Total Suspended Solids
ug/l	micrograms per liter (equal to ppb)
VMT	Vehicle Miles Traveled
VOC	Volatile Organic Compound
WLF	Water Level Fluctuation
WTP	Wastewater Treatment Plant

Chapter 1: Introduction

This research monograph comprehensively reviews the available scientific data on the impacts of urbanization on small streams and receiving waters. These impacts are generally classified according to one of four broad categories: changes in hydrologic, physical, water quality or biological indicators. More than 225 research studies have documented the adverse impact of urbanization on one or more of these key indicators. In general, most research has focused on smaller watersheds, with drainage areas ranging from a few hundred acres up to ten square miles.

Streams vs. Downstream Receiving Waters

Urban watershed research has traditionally pursued two core themes. One theme has evaluated the direct impact of urbanization on small streams, whereas the second theme has explored the more indirect impact of urbanization on downstream receiving waters, such as rivers, lakes, reservoirs, estuaries and coastal areas. This report is organized to profile recent research progress in both thematic areas and to discuss the implications each poses for urban watershed managers.

When evaluating the direct impact of urbanization on streams, researchers have emphasized hydrologic, physical and biological indicators to define urban stream quality. In recent years, impervious cover (IC) has emerged as a key paradigm to explain and sometimes predict how severely these stream quality indicators change in response to different levels of watershed development. The Center for Watershed Protection has integrated these research findings into a general watershed planning model, known as the impervious cover model (ICM). The ICM predicts that most stream quality indicators decline when watershed IC exceeds 10%, with severe degradation expected beyond 25% IC. In the first part of this review, we critically analyze the scientific basis for the ICM and explore some of its more interesting technical implications.

While many researchers have monitored the quality of stormwater runoff from small watersheds, few have directly linked these pollutants to specific water quality problems within streams (e.g., toxicity, biofouling, eutrophication). Instead, the prevailing view is that stormwater pollutants are a downstream export. That is, they primarily influence downstream receiving water quality. Therefore, researchers have focused on how to estimate stormwater pollutant loads and then determine the water quality response of the rivers, lakes and estuaries that receive them. To be sure, there is an increasing recognition that runoff volume can influence physical and biological indicators within some receiving waters, but only a handful of studies have explored this area. In the second part of this review, we review the impacts of urbanization on downstream receiving waters, primarily from the standpoint of stormwater quality. We also evaluate whether the ICM can be extended to predict water quality in rivers, lakes and estuaries.

This chapter is organized as follows:

1.1 A Review of Recent Urban Stream Research and the ICM
1.2 Impacts of Urbanization on Downstream Receiving Waters
1.3 Implications of the ICM for Watershed Managers

1.1 A Review of Recent Urban Stream Research and the ICM

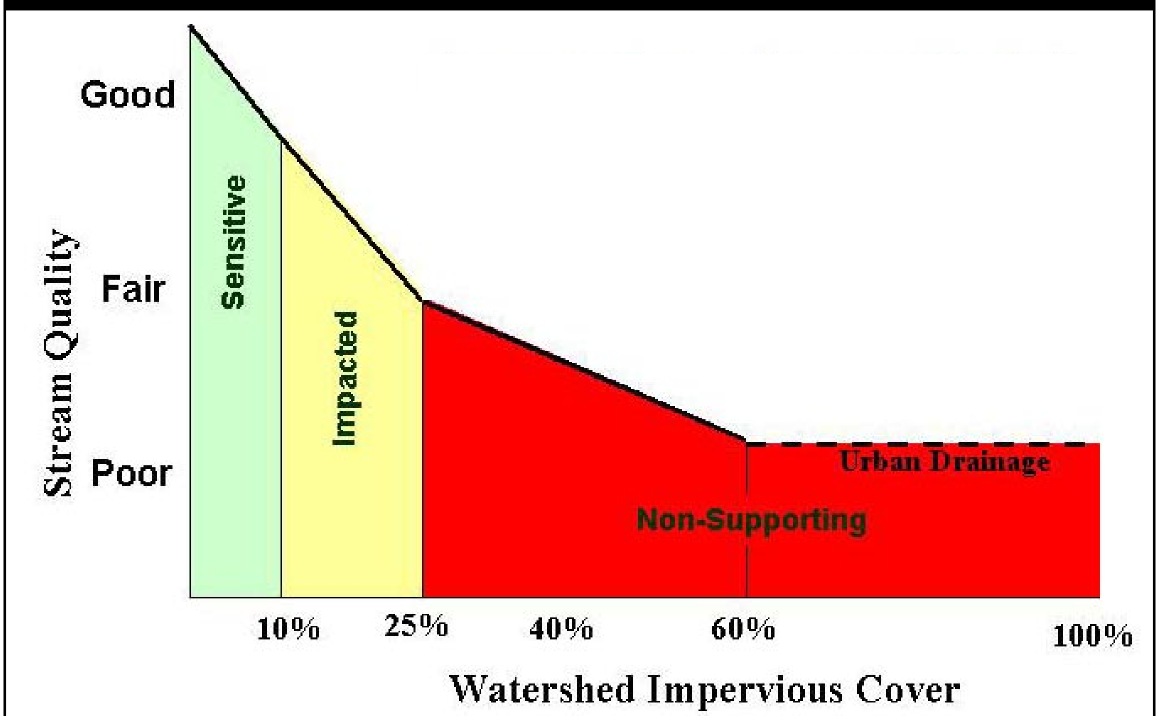
In 1994, the Center published "The Importance of Imperviousness," which outlined the scientific evidence for the relationship between IC and stream quality. At that time, about two dozen research studies documented a reasonably strong relationship between watershed IC and various indicators of stream quality. The research findings were subsequently integrated into the ICM (Schueler, 1994a and CWP, 1998). A brief summary of the basic assumptions of the ICM can be found in Figure 1. The ICM has had a major influence in watershed planning, stream classification and land use regulation in many communities. The ICM is a deceptively simple model that raises extremely complex and profound policy implications for watershed managers.

The ICM has been widely applied in many urban watershed settings for the purposes of small watershed planning, stream classification, and supporting restrictive development regulations and watershed zoning. As such, the ICM has stimulated intense debate among the planning, engineering and scientific communities. This debate is likely to soon spill over into the realm of politics and the courtroom, given its potential implications for local land use and environmental regulation. It is no wonder that the specter of scientific uncertainty is frequently invoked in the ICM debate, given the land use policy issues at stake. In this light, it is helpful to review the current strength of the evidence for and against the ICM.

The ICM is based on the following assumptions and caveats:

- Applies only to 1st, 2nd and 3rd order streams.
- Requires accurate estimates of percent IC, which is defined as the total amount of impervious cover over a subwatershed area.
- Predicts <u>potential</u> rather than <u>actual</u> stream quality. It can and should be expected that some streams will depart from the predictions of the model. For example, monitoring indicators may reveal poor water quality in a stream classified as "sensitive" or a surprisingly high biological diversity

Figure 1: Impervious Cover Model



score in a "non-supporting" one. Consequently, while IC can be used to initially diagnose stream quality, supplemental field monitoring is recommended to actually confirm it.

- Does not predict the precise score of an individual stream quality indicator but rather predicts the average behavior of a group of indicators over a range of IC. Extreme care should be exercised if the ICM is used to predict the fate of individual species (e.g., trout, salmon, mussels).

- "Thresholds" defined as 10 and 25% IC are not sharp "breakpoints," but instead reflect the expected transition of a composite of individual indicators in that range of IC. Thus, it is virtually impossible to distinguish real differences in stream quality indicators within a few percentage points of watershed IC (e.g., 9.9 vs. 10.1%).

- Should only be applied within the ecoregions where it has been tested, including the mid-Atlantic, Northeast, Southeast, Upper Midwest, and Pacific Northwest.

- Has not yet been validated for non-stream conditions (e.g., lakes, reservoirs, aquifers and estuaries).

- Does not currently predict the impact of watershed treatment.

In this section, we review available stream research to answer four questions about the ICM:

1. Does recent stream research still support the basic ICM?
2. What, if any, modifications need to be made to the ICM?
3. To what extent can watershed practices shift the predictions of the ICM?
4. What additional research is needed to test the ICM?

1.1.1 Strength of the Evidence for the ICM

Many researchers have investigated the IC/stream quality relationship in recent years. The Center recently undertook a comprehensive analysis of the literature to assess the scientific basis for the ICM. As of the end of 2002, we discovered more than 225 research studies that measured 26 different urban stream indicators within many regions of North America. We classified the research studies into three basic groups.

The first and most important group consists of studies that directly test the IC/stream quality indicator relationship by monitoring a large population of small watersheds. The second and largest group encompasses secondary studies that indirectly support the ICM by showing significant differences in stream quality indicators between urban and non-urban watersheds. The third and last group of studies includes widely accepted engineering models that explicitly use IC to directly predict stream quality indicators. Examples include engineering models that predict peak discharge or stormwater pollutant loads as a direct function of IC. In most cases, these relationships were derived from prior empirical research.

Table 1 provides a condensed summary of recent urban stream research, which shows the impressive growth in our understanding of urban streams and the watershed factors that influence them. A negative relationship between watershed development and nearly all of the 26 stream quality indicators has been established over many regions and scientific disciplines. About 50 primary studies have tested the IC/stream quality indicator relationship, with the largest number looking at biological indicators of stream health, such as the diversity of aquatic insects or fish. Another 150 or so secondary studies provide evidence that stream quality indicators are significantly different between urban and non-urban watersheds, which lends at least indirect support for the ICM and suggests that additional research to directly test the IC/stream quality indicator

Table 1: The Strength of Evidence: A Review of the Current Research on Urban Stream Indicators						
Stream Quality Indicator	**#**	**IC**	**UN**	**EM**	**RV**	**Notes**
Increased Runoff Volume	2	Y	Y	Y	N	extensive national data
Increased Peak Discharge	7	Y	Y	Y	Y	type of drainage system key
Increased Frequency of Bankfull Flow	2	?	Y	N	N	hard to measure
Diminished Baseflow	8	?	Y	N	Y	inconclusive data
Stream Channel Enlargement	8	Y	Y	N	Y	stream type important
Increased Channel Modification	4	Y	Y	N	?	stream enclosure
Loss of Riparian Continuity	4	Y	Y	N	?	can be affected by buffer
Reduced Large Woody Debris	4	Y	Y	N	?	Pacific NW studies
Decline in Stream Habitat Quality	11	Y	Y	N	?	
Changes in Pool Riffle/Structure	4	Y	Y	N	?	
Reduced Channel Sinuosity	1	?	Y	N	?	straighter channels
Decline in Streambed Quality	2	Y	Y	N	?	embeddedness
Increased Stream Temperature	5	Y	Y	N	?	buffers and ponds also a factor
Increased Road Crossings	3	?	Y	N	?	create fish barriers
Increased Nutrient Load	30+	?	Y	Y	N	higher stormwater EMCs
Increased Sediment Load	30+	?	Y	N	Y	higher EMCs in arid regions
Increased Metals & Hydrocarbons	20+	?	Y	Y	N	related to traffic/VMT
Increased Pesticide Levels	7	?	Y	N	Y	may be related to turf cover
Increased Chloride Levels	5	?	Y	N	Y	related to road density
Violations of Bacteria Standards	9	Y	Y	N	Y	indirect association
Decline in Aquatic Insect Diversity	33	Y	Y	N	N	IBI and EPT
Decline in Fish Diversity	19	Y	Y	N	N	regional IBI differences
Loss of Coldwater Fish Species	6	Y	Y	N	N	trout and salmon
Reduced Fish Spawning	3	Y	Y	N	?	
Decline in Wetland Plant Diversity	2	N	Y	N	?	water level fluctuation
Decline in Amphibian Community	5	Y	Y	N	?	few studies

#: *total number of all studies that evaluated the indicator for urban watersheds*
IC: *does balance of studies indicate a progressive change in the indicator as IC increases? Answers: Yes, No or No data (?)*
UN: *If the answer to IC is no, does the balance of the studies show a change in the indicator from non-urban to urban watersheds? Yes or No*
EM *Is the IC/stream quality indicator relationship implicitly assumed within the framework of widely accepted engineering models? Yes, No or No models yet exist (?)*
RV: *If the relationship has been tested in more than one eco-region, does it generally show major differences between ecoregions? Answers: Yes, No, or insufficient data (?)*

relationship is warranted. In some cases, the IC/stream quality indicator relationship is considered so strongly established by historical research that it has been directly incorporated into accepted engineering models. This has been particularly true for hydrological and water quality indicators.

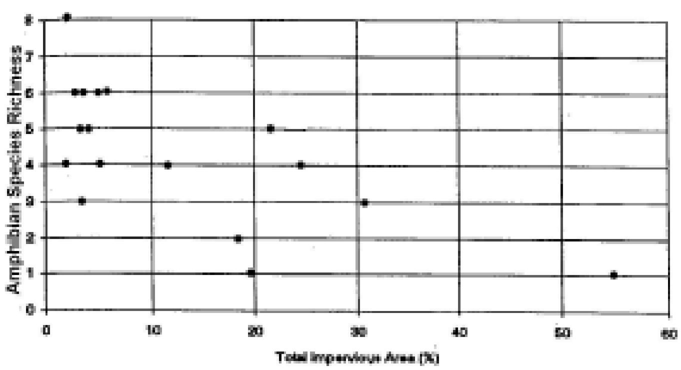
1.1.2 Reinterpretation of the ICM

Although the balance of recent stream research generally supports the ICM, it also offers several important insights for interpreting and applying the ICM, which are discussed next.

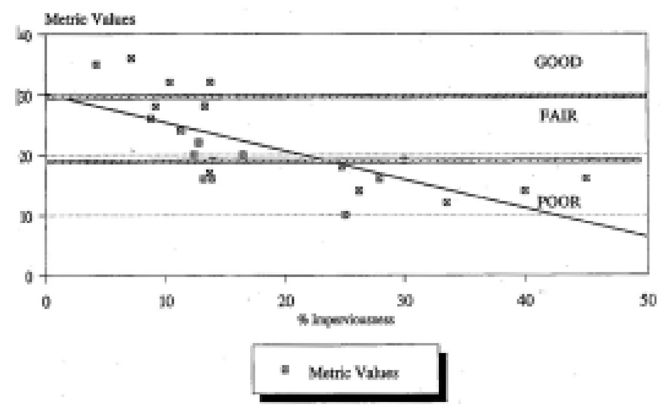
Statistical Variability

Scatter is a common characteristic of most IC/stream quality indicator relationships. In most cases, the overall trend for the indicator is down, but considerable variation exists along the trend line. Often, linear regression equations between IC and individual stream quality indicators produce relatively modest correlation coefficients (reported r^2 of 0.3 to 0.7 are often considered quite strong).

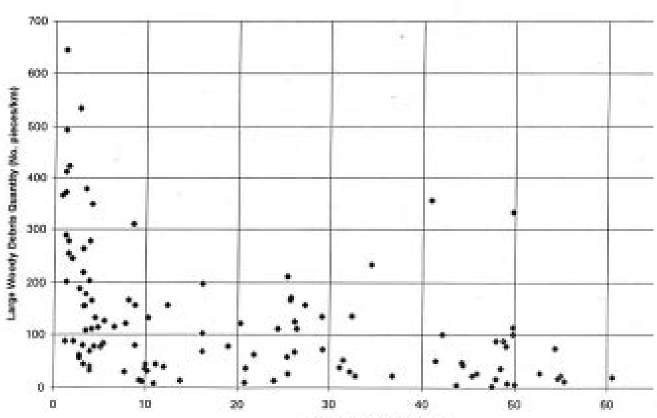
Figure 2 shows typical examples of the IC/stream quality indicator relationship that illustrate the pattern of statistical variability. Variation is always encountered when dealing with urban stream data (particularly so for biological indicators), but several patterns exist that have important implications for watershed managers.



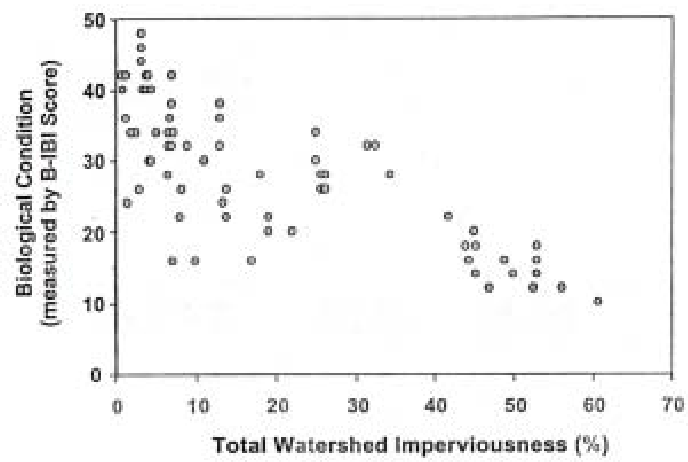
a. Fish IBI vs. IC in Fairfax, VA (Fairfax County, 2001)



b. CPSS vs. IC in Montgomery County, MD (MNCPPC, 2000)



c. Large Woody Debris vs. IC (Booth *et al.*, 1997)



d. Biological Condition vs. Total Watershed IC (Booth, 2000)

Figure 2: Typical Scatter Found in IC/Stream Quality Indicator Research

The first pattern to note is that the greatest scatter in stream quality indicator scores is frequently seen in the range of one to 10% IC. These streams, which are classified as "sensitive" according to the ICM, often exhibit low, moderate or high stream quality indicator scores, as shown in Figure 2. The key interpretation is that sensitive streams have the potential to attain high stream quality indicator scores, but may not always realize this potential.

Quite simply, the influence of IC in the one to 10% range is relatively weak compared to other potential watershed factors, such as percent forest cover, riparian continuity, historical land use, soils, agriculture, acid mine drainage or a host of other stressors. Consequently, watershed managers should never rely on IC alone to classify and manage streams in watersheds with less than 10% IC. Rather, they should evaluate a range of supplemental watershed variables to measure or predict actual stream quality within these lightly developed watersheds.

The second important pattern is that variability in stream quality indicator data is usually dampened when IC exceeds 10%, which presumably reflects the stronger influence of stormwater runoff on stream quality indicators. In particular, the chance that a stream quality indicator will attain a high quality score is sharply diminished at higher IC levels. This trend becomes pronounced within the 10 to 25% IC range and almost inevitable when watershed IC exceeds 25%. Once again, this pattern suggests that IC is a more robust and reliable indicator of overall stream quality beyond the 10% IC threshold.

Other Watershed Variables and the ICM
Several other watershed variables can potentially be included in the ICM. They include forest cover, riparian forest continuity and turf cover.

Forest cover (FC) is clearly the main rival to IC as a useful predictor of stream quality in urban watersheds, at least for humid regions of North America. In some regions, FC is simply the reciprocal of IC. For example, Horner and May (1999) have demonstrated a strong interrelationship between IC and FC for subwatersheds in the Puget Sound region (Figure 3). In other regions, however, "pre-

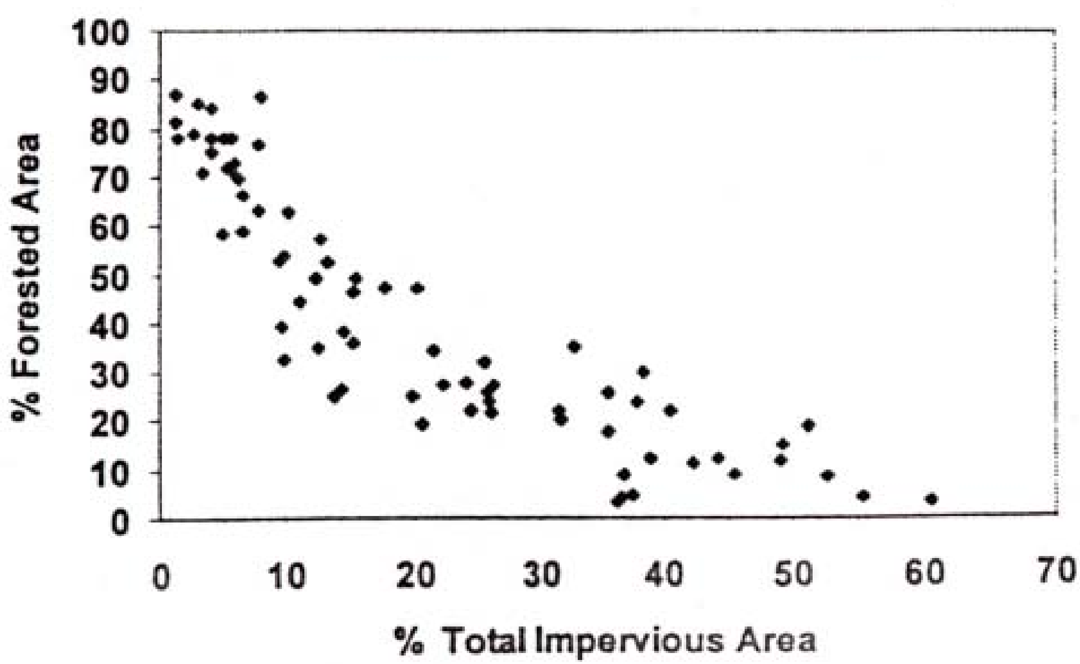


Figure 3: Relationship of IC and FC in Puget Sound Subwatersheds (Horner and May, 1999)

development" land use represents a complex mosaic of crop land, pasture and forest. Therefore, an inverse relationship between FC and IC may not be universal for subwatersheds that have witnessed many cycles of deforestation and cultivation.

It should come as little surprise that the progressive loss of FC has been linked to declining stream quality indicators, given that forested watersheds are often routinely used to define natural reference conditions for streams (Booth, 2000 and Horner *et al.*, 2001). Mature forest is considered to be the main benchmark for defining pre-development hydrology within a subwatershed, as well. Consequently, FC is perhaps the most powerful indicator to predict the quality of streams within the "sensitive" category (zero to 10% IC).

To use an extreme example, one would expect that stream quality indicators would respond quite differently in a subwatershed that had 90% FC compared to one that had 90% crop cover. Indeed, Booth (1991) suggests that stream quality can only be maintained when IC is limited to less than 10% <u>and</u> at least 65% FC is retained within a subwatershed. The key management implication then is that stream health is best managed by simultaneously minimizing the creation of IC and maximizing the preservation of native FC.

FC has also been shown to be useful in predicting the quality of terrestrial variables in a subwatershed. For example, the Mid-Atlantic Integrated Assessment (USEPA, 2000) has documented that watershed FC can reliably predict the diversity of bird, reptile and amphibian communities in the mid-Atlantic region. Moreover, the emerging discipline of landscape ecology provides watershed managers with a strong scientific foundation for deciding where FC should be conserved in a watershed. Conservation plans that protect and connect large forest fragments have been shown to be effective in conserving terrestrial species.

Riparian forest continuity has also shown considerable promise in predicting at least some indicators of stream quality for urban watersheds. Researchers have yet to come up with a standard definition of riparian continuity, but it is usually defined as the proportion of the perennial stream network in a subwatershed that has a fixed width of mature streamside forest. A series of studies indicates that aquatic insect and fish diversity are associated with high levels of riparian continuity (Horner *et al.,* 2001; May *et al.*, 1997; MNCPPC, 2000; Roth *et al.,* 1998). On the other hand, not much evidence has been presented to support the notion that riparian continuity has a strong influence on hydrology or water quality indicators.

One watershed variable that received little attention is the fraction of watershed area maintained in turf cover (TC). Grass often comprises the largest fraction of land area within low-density residential development and could play a significant role in streams that fall within the "impacted" category (10 to 25% IC). Although lawns are pervious, they have sharply different properties than the forests and farmlands they replace (i.e., irrigation, compacted soils, greater runoff, and much higher input of fertilizers and pesticides, etc.). It is interesting to speculate whether the combined area of IC and TC might provide better predictions about stream health than IC area alone, particularly within impacted subwatersheds.

Several other watershed variables might have at least supplemental value in predicting stream quality. They include the presence of extensive wetlands and/or beaverdam complexes in a subwatershed; the dominant form of drainage present in the watershed (tile drains, ditches, swales, curb and gutters, storm drain pipes); the average age of development; and the proximity of sewer lines to the stream. As far as we could discover, none of these variables has been systematically tested in a controlled population of small watersheds. We have observed that these factors could be important in our field investigations and often measure them to provide greater insight into subwatershed behavior.

Lastly, several watershed variables that are closely related to IC have been proposed to predict stream quality. These include population, percent urban land, housing density, road density and other indices of watershed development. As might be expected, they generally track the same trend as IC, but each has some significant technical limitations and/or difficulties in actual planning applications (Brown, 2000).

Individual vs. Multiple Indicators

The ICM does not predict the precise score of individual stream quality indicators, but rather predicts the average behavior of a group of indicators over a range of IC. Extreme care should be exercised if the ICM is used to predict the fate of individual indicators and/or species. This is particularly true for sensitive aquatic species, such as trout, salmon, and freshwater mussels. When researchers have examined the relationship between IC and individual species, they have often discovered lower thresholds for harm. For example, Boward *et al.* (1999) found that brook trout were not found in subwatersheds that had more than 4% IC in Maryland, whereas Horner and May (1999) asserted an 8% threshold for sustaining salmon in Puget Sound streams.

The key point is that if watershed managers want to maintain an individual species, they should be very cautious about adopting the 10% IC threshold. The essential habitat requirements for many sensitive or endangered species are probably determined by the *most sensitive* stream quality indicators, rather than the *average behavior* of all stream quality indicators.

Direct Causality vs. Association

A strong relationship between IC and declining stream quality indicators does not always mean that the IC is directly responsible for the decline. In some cases, however, causality can be demonstrated. For example, increased stormwater runoff volumes are directly caused by the percentage of IC in a subwatershed, although other factors such as conveyance, slope and soils may play a role.

In other cases, the link is much more indirect. For these indicators, IC is merely an index of the cumulative amount of watershed development, and more IC simply means that a greater number of known or unknown pollutant sources or stressors are present. In yet other cases, a causal link appears likely but has not yet been scientifically demonstrated. A good example is the more than 50 studies that have explored how fish or aquatic insect diversity changes in response to IC. While the majority of these studies consistently shows a very strong negative association between IC and biodiversity, they do not really establish which stressor or combination of stressors contributes most to the decline. The widely accepted theory is that IC changes stream hydrology, which degrades stream habitat, and in turn leads to reduced stream biodiversity.

Regional Differences

Currently, the ICM has been largely confirmed within the following regions of North America: the mid-Atlantic, the Northeast, the Southeast, the upper Midwest and the Pacific Northwest. Limited testing in Northern California, the lower Midwest and Central Texas generally agrees with the ICM. The ICM has not been tested in Florida, the Rocky Mountain West, and the Southwest. For a number of reasons, it is not certain if the ICM accurately predicts biological indicators in arid and semiarid climates (Maxted, 1999).

Measuring Impervious Cover

Most researchers have relied on total impervious cover as the basic unit to measure IC at the subwatershed level. The case has repeatedly been made that effective impervious cover is probably a superior metric (e.g., only counting IC that is hydraulically connected to the drainage system). Notwithstanding, most researchers have continued to measure total IC because it is generally quicker and does not require extensive (and often subjective) engineering judgement as to whether it is connected or not. Researchers have used a wide variety of techniques to estimate subwatershed IC, including satellite imagery, analysis of aerial photographs, and derivation from GIS land use layers. Table 2 presents some standard land use/IC relationships that were developed for suburban regions of the Chesapeake Bay.

Table 2: Land Use/IC Relationships for Suburban Areas of the Chesapeake Bay (Cappiella and Brown, 2001)					
Land Use Category	**Sample Number (N)**	**Mean IC (SE)**	**Land Use Category**	**Sample Number (N)**	**Mean IC (SE)**
Agriculture	8	**1.9** – 0.3	Institutional	30	**34.4** – 3.45
Open Urban Land	11	**8.6** – 1.64	Light	20	**53.4** – 2.8
2 Acre Lot Residential	12	**10.6** – 0.65	Commercia	23	**72.2** – 2.0
1 Acre Lot Residential	23	**14.3** – 0.53	Churches	8	**39.9** – 7.8 1
1/2 Acre Lot Residential	20	**21.2** – 0.78	Schools	13	**30.3** – 4.8
1/4 Acre Lot Residential	23	**27.8** – 0.60	Municipals	9	**35.4** – 6.3
1/8 Acre Lot Residential	10	**32.6** – 1.6	Golf	4	**5.0** – 1.7
Townhome Residential	20	**40.9** – 1.39	Cemeteries	3	**8.3** – 3.5
Multifamily Residential	18	**44.4** – 2.0	Parks	4	**12.5** – 0.7

Three points are worth noting. First, it is fair to say that most researchers have spent more quality control effort on their stream quality indicator measurements than on their subwatershed IC estimates. At the current time, no standard protocol exists to estimate subwatershed IC, although Cappiella and Brown (2001) presented a useful method. At best, the different methods used to measure IC make it difficult to compare results from different studies, and at worst, it can introduce an error term of perhaps +/- 10% from the true value within an individual subwatershed. Second, it is important to keep in mind that IC is not constant over time; indeed, major changes in subwatershed IC have been observed within as few as two years. Consequently, it is sound practice to obtain subwatershed IC estimates from the most recent possible mapping data, to ensure that it coincides with stream quality indicator measurements. Lastly, it is important to keep in mind that most suburban and even rural zoning categories exceed 10% IC (see Table 2). Therefore, from a management standpoint, planners should try to project future IC, in order to determine the future stream classification for individual subwatersheds.

1.1.3 Influence of Watershed Treatment Practices on the ICM

The most hotly debated question about the ICM is whether widespread application of watershed practices such as stream buffers or stormwater management can mitigate the impact of IC, thereby allowing greater development density for a given watershed. At this point in time, there are fewer than 10 studies that directly bear on this critical question. Before these are reviewed, it is instructive to look at the difficult technical and scientific issues involved in detecting the effect of watershed treatment, given its enormous implications for land use control and watershed management.

The first tough issue is how to detect the effect of watershed treatment, given the inherent scatter seen in the IC/stream quality indicator relationship. Figure 4 illustrates the “double scatter” problem, based on three different urban stream research studies in Delaware, Maryland and Washington. A quick inspection of the three plots shows how intrinsically hard it is to distinguish the watershed treatment effect. As can be seen, stream quality indicators in subwatersheds with treatment tend to

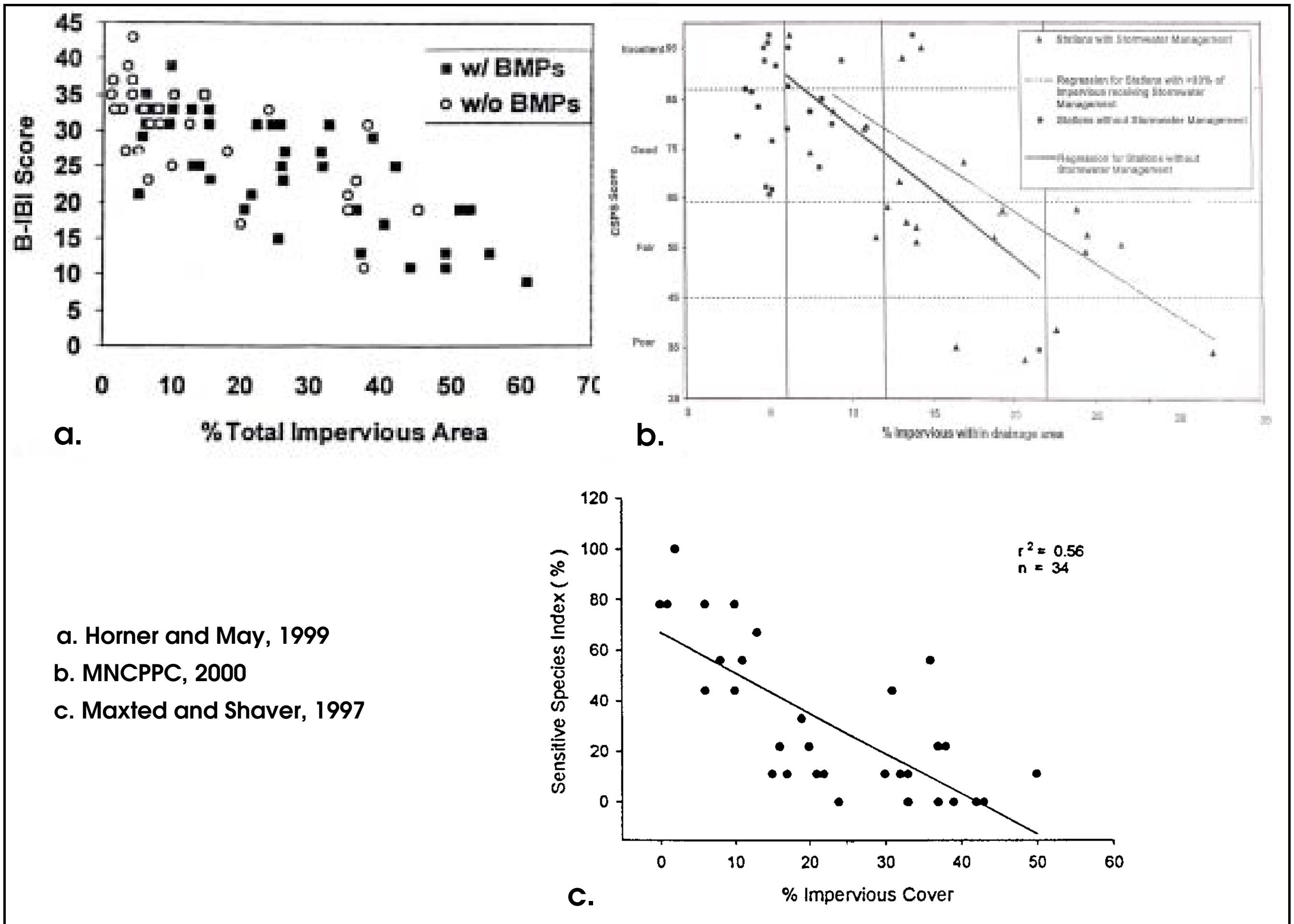


Figure 4: The Double Scatter Problem: Difficulties in Detecting the Effect of Watershed Treatment

overplot those in subwatersheds that lack treatment. While subtle statistical differences may be detected, they are not visibly evident. This suggests that the impact of watershed treatment would need to be extremely dramatic to be detected, given the inherent statistical variability seen in small watersheds (particularly so within the five to 25% IC range where scatter is considerable).

In an ideal world, a watershed study design would look at a controlled population of small urban watersheds that were developed with and without watershed practices to detect the impact of "treatment." In the real world, however, it is impossible to strictly control subwatershed variables. Quite simply, no two subwatersheds are ever alike. Each differs slightly with respect to drainage area, IC, forest cover, riparian continuity, historical land use, and percent watershed treatment. Researchers must also confront other real world issues when designing their watershed treatment experiments.

For example, researchers must carefully choose which indicator or group of indicators will be used to define stream health. IC has a negative influence on 26 stream quality indicators, yet nearly all of the watershed treatment research so far has focused on just a few biological indicators (e.g., aquatic insect or fish diversity) to define stream health. It is conceivable that watershed treatment might have no effect on biological indicators, yet have a positive influence on hydrology, habitat or water quality indicators. At this point, few of these indicators have been systematically tested in the field. It is extremely doubtful that any watershed practice can simultaneously improve or mitigate all 26 stream quality indicators, so researchers must carefully interpret the outcomes of their watershed treatment experiments.

The second issue involves how to quantify watershed treatment. In reality, watershed treatment collectively refers to dozens of practices that are installed at individual development sites in the many years or even decades it takes to fully "build out" a subwatershed. Several researchers have discovered that watershed practices are seldom installed consistently across an entire subwatershed. In some cases, less than a third of the IC in a subwatershed was actually treated by any practice, because development occurred prior to regulations; recent projects were exempted, waived or grandfathered; or practices were inadequately constructed or maintained (Horner and May, 1999 and MNCPPC, 2000).

Even when good coverage is achieved in a watershed, such as the 65 to 90% reported in studies of stormwater ponds (Jones *et al*., 1996; Maxted, 1999; Maxted and Shaver, 1997), it is still quite difficult to quantify the actual quality of treatment. Often, each subwatershed contains its own unique mix of stormwater practices installed over several decades, designed under diverse design criteria, and utilizing widely different stormwater technologies. Given these inconsistencies, researchers will need to develop standard protocols to define the extent and quality of watershed treatment.

Effect of Stormwater Ponds

With this in mind, the effect of stormwater ponds and stream buffers can be discussed. The effect of larger stormwater ponds in mitigating the impacts of IC in small watersheds has received the most scrutiny to date. This is not surprising, since larger ponds often control a large fraction of their contributing subwatershed area (e.g. 100 to 1,000 acres) and are located on the stream itself, therefore lending themselves to easier monitoring. Three studies have evaluated the impact of large stormwater ponds on downstream aquatic insect communities (Jones *et al*., 1996; Maxted and Shaver, 1997; Stribling *et al*., 2001). Each of these studies was conducted in small headwater subwatersheds in the mid-Atlantic Region, and none was able to detect major differences in aquatic insect diversity in streams with or without stormwater ponds.

Four additional studies statistically evaluated the stormwater treatment effect in larger populations of small watersheds with varying degrees of IC (Horner and May, 1999; Horner *et al.,* 2001; Maxted, 1999; MNCPPC, 2000). These studies generally sampled larger watersheds that had many stormwater practices but not necessarily complete watershed coverage. In general, these studies detected a small but positive effect of stormwater treatment relative to aquatic insect diversity. This positive effect was typically seen only in the range of five to 20% IC and was generally undetected beyond about 30% IC. Although each author was hesitant about interpreting his results, all generally agreed that perhaps as much as 5% IC could be added to a subwatershed while maintaining aquatic insect diversity, given effective stormwater treatment. Forest retention and stream buffers were found to be very important, as well. Horner *et al.* (2001) reported a somewhat stronger IC threshold for various species of salmon in Puget Sound streams.

Some might conclude from these initial findings that stormwater ponds have little or no value in maintaining biological diversity in small streams. However, such a conclusion may be premature for several reasons. First, the generation of stormwater ponds that was tested was not explicitly designed to protect stream habitat or to prevent downstream channel erosion, which would presumably promote aquatic diversity. Several states have recently changed their stormwater criteria to require extended detention for the express purpose of preventing downstream channel erosion, and these new criteria may exert a stronger influence on aquatic diversity. Instead, their basic design objective was to maximize pollutant removal, which they did reasonably well.

The second point to stress is that streams with larger stormwater ponds should be considered “regulated streams” (Ward and Stanford, 1979), which have a significantly altered aquatic insect community downstream of the ponds. For example, Galli (1988) has reported that on-stream wet stormwater ponds shift the trophic structure of the aquatic insect community. The insect community above the pond was dominated by shredders, while the insect community below the pond was dominated by scrapers, filterers and collectors. Of particular note, several pollution-sensitive species were eliminated below the pond. Galli reported that changes in stream temperatures, carbon supply and substrate fouling were responsible for the downstream shift in the aquatic insect community. Thus, while it is clear that large stormwater ponds can be expected to have a negative effect on aquatic insect diversity, they could still exert positive influence on other stream quality indicators.

Effect of Stream Buffers

A handful of studies have evaluated biological indicator scores for urban streams that have extensive forest buffers, compared to streams where they were mostly or completely absent (Horner and May, 1999; Horner *et al.,* 2001; May *et al.*, 1997; MNCPPC, 2000; Roth *et al.,* 1998; Steedman, 1988). Biological indicators included various indices of aquatic insect, fish and salmon diversity. Each study sampled a large population of small subwatersheds over a range of IC and derived a quantitative measure to express the continuity, width and forest cover of the riparian buffer network within each subwatershed. Riparian forests were hypothesized to have a positive influence on stream biodiversity, given the direct ways they contribute to stream habitat (e.g., shading, woody debris, leaf litter, bank stability, and organic carbon supply).

All five studies detected a small to moderate positive effect when forested stream buffers were present (frequently defined as at least two-thirds of the stream network with at least 100 feet of stream side forest). The greatest effect was reported by Horner and May (1999) and Horner *et al.* (2001) for salmon streams in the Puget Sound ecoregion. If excellent riparian habitats were preserved, they generally reported that fish diversity could be maintained up to 15% IC, and good aquatic insect diversity could be maintained with as much as 30% IC. Steedman (1988) reported a somewhat smaller effect for Ontario streams. MNCPPC (2000), May *et al.* (1997), and Roth *et al.* (1998) could not find a statistically significant relationship between riparian quality and urban stream quality indicators but did report that most outliers (defined as higher IC subwatersheds with unusually high biological indicator scores) were generally associated with extensive stream side forest.

1.1.4 Recommendations for Further ICM Research

At this point, we recommend three research directions to improve the utility of the ICM for watershed managers. The **first direction** is to expand basic research on the relationship between IC and stream quality indicators that have received little scrutiny. In particular, more work is needed to define the relationship between IC and hydrological and physical indicators such as the following:

- Physical loss or alteration of the stream network
- Stream habitat measures
- Riparian continuity
- Baseflow conditions during dry weather

In addition, more watershed research is needed in ecoregions and physiographic areas where the ICM has not yet been widely tested. Key areas include Florida, arid and semiarid climates, karst areas and mountainous regions. The basic multiple subwatershed monitoring protocol set forth by Schueler (1994a) can be used to investigate IC/stream quality relationships, although it would be wise to measure a wider suite of subwatershed variables beyond IC (e.g., forest cover, turf cover, and riparian continuity).

The **second** research direction is to more clearly define the impact of watershed treatment on stream quality indicators. Based on the insurmountable problems encountered in controlling variation at the subwatershed level, it may be necessary to abandon the multiple watershed or paired watershed sampling approaches that have been used to date. Instead, longitudinal monitoring studies within individual subwatersheds may be a more powerful tool to detect the effect of watershed treatment. These studies could track changes in stream quality indicators in individual subwatersheds over the entire development cycle: pre-development land use, clearing, construction, build out, and post construction. In most cases, longitudinal studies would take five to 10 years to complete, but they would allow watershed managers to measure and control the inherent variability at the subwatershed level and provide a “before and after” test of watershed treatment. Of course, a large population of test subwatersheds would be needed to satisfactorily answer the watershed treatment question.

The **third** research direction is to monitor more non-supporting streams, in order to provide a stronger technical foundation for crafting more realistic urban stream standards and to see how they respond to various watershed restoration treatments. As a general rule, most researchers have been more interested in the behavior of sensitive and impacted streams. The non-supporting stream category spans a wide range of IC, yet we do not really understand how stream quality indicators behave over the entire 25 to 100% IC range.

For example, it would be helpful to establish the IC level at the upper end of the range where streams are essentially transformed into an artificial conveyance system (i.e., become pipes or artificial channels). It would also be interesting to sample more streams near the lower end of the non-supporting category (25 to 35% IC) to detect whether stream quality indicators respond to past watershed treatment or current watershed restoration efforts. For practical reasons, the multiple subwatershed sampling approach is still recommended to characterize indicators in non-supporting streams. However, researchers will need to screen a large number of non-supporting subwatersheds in order to identify a few subwatersheds that are adequate for subsequent sampling (i.e., to control for area, IC, development age, percent watershed treatment, type of conveyance systems, etc.).

1.2 Impacts of Urbanization on Downstream Receiving Waters

In this section, we review the impacts of urbanization on downstream receiving waters, primarily from the standpoint of impacts caused by poor stormwater quality. We begin by looking at the relationship between IC and stormwater pollutant loadings. Next, we discuss the sensitivity of selected downstream receiving waters to stormwater pollutant loads. Lastly, we examine the effect of watershed treatment in reducing stormwater pollutant loads.

1.2.1 Relationship Between Impervious Cover and Stormwater Quality

Urban stormwater runoff contains a wide range of pollutants that can degrade downstream water quality (Table 3). Several generalizations can be supported by the majority of research conducted to date. First, the unit area pollutant load delivered by stormwater runoff to receiving waters increases in direct proportion to watershed IC. This is not altogether surprising, since pollutant load is the product of the average pollutant concentration and stormwater runoff volume. Given that runoff volume increases in direct proportion to IC, pollutant loads must automatically increase when IC increases, as long the average pollutant concentration stays the same (or increases). This relationship is a central assumption in most simple and complex pollutant loading models (Bicknell *et al.*, 1993; Donigian and Huber, 1991; Haith *et al.*, 1992; Novotny and Chester, 1981; NVPDC, 1987; Pitt and Voorhees, 1989).

The second generalization is that stormwater pollutant concentrations are generally similar

Table 3: Summary of Urban Stormwater Pollutant Loads on Quality of Receiving Waters

Pollutants in Urban Stormwater	WQ Impacts To:					Higher Unit Load?	Load a function of IC?	Other Factors Important in Loading
	R	L	E	A	W			
Suspended Sediment	Y	Y	Y	N	Y	Y (ag)	Y	channel erosion
Total Nitrogen	N	N	Y	Y	N	Y (ag)	Y	septic systems
Total Phosphorus	Y	Y	N	N	Y	Y (ag)	Y	tree canopy
Metals	Y	Y	Y	?	N	Y	Y	vehicles
Hydrocarbons	Y	Y	Y	Y	Y	Y	?	related to VMTs and hotspots
Bacteria/Pathogens	Y	Y	Y	N	Y	Y	Y	many sources
Organic Carbon	N	?	?	?	Y	Y	Y	
MTBE	N	N	N	Y	Y	Y	?	roadway, VMTs
Pesticides	?	?	?	?	Y	Y	?	turf/landscaping
Chloride	?	Y	N	Y	Y	Y	?	road density
Trash/Debris	Y	Y	Y	N	?	Y	Y	curb and gutters

Major Water Quality Impacts Reported for:
R = River, L = Lake, E = Estuary, A = Aquifer, W = Surface Water Supply
Higher Unit Area Load? Yes (compared to all land uses) (ag): with exception of cropland
Load a function of IC? Yes, increases proportionally with IC

at the catchment level, regardless of the mix of IC types monitored (e.g., residential, commercial, industrial or highway runoff). Several hundred studies have examined stormwater pollutant concentrations from small urban catchments and have generally found that the variation within a catchment is as great as the variation between catchments. Runoff concentrations tend to be log-normally distributed, and therefore the long term "average" concentration is best expressed by a median value. It should be kept in mind that researchers have discovered sharp differences in pollutant concentrations for smaller, individual components of IC (e.g., rooftops, parking lots, streets, driveways and the like). Since most urban catchments are composed of many kinds of IC, this mosaic quality tempers the variability in long term pollutant concentrations at the catchment or subwatershed scale.

The third generalization is that median concentrations of pollutants in urban runoff are usually higher than in stormwater runoff from most other non-urban land uses. Consequently, the unit area nonpoint pollutant load generated by urban land normally exceeds that of nearly all watershed land uses that it replaces (forest, pasture, cropland, open space — see Table 3). One important exception is cropland, which often produces high unit area sediment and nutrient loads in many regions of the country. In these watersheds, conversion of intensively managed crops to low density residential development may actually result in a slightly decreased sediment or nutrient load. On the other hand, more intensive land development (30% IC or more) will tend to equal or exceed cropland loadings.

The last generalization is that the effect of IC on stormwater pollutant loadings tends to be weakest for subwatersheds in the one to 10% IC range. Numerous studies have suggested that other watershed and regional factors may have a stronger influence, such as the underlying geology, the amount of carbonate rock in the watershed, physiographic region, local soil types, and most important, the relative fraction of forest and crop cover in the subwatershed (Herlihy *et al*., 1998 and Liu *et al.,* 2000). The limited influence of IC on pollutant loads is generally consistent with the finding for hydrologic, habitat and biological indicators over this narrow range of IC. Once again, watershed managers are advised to track other watershed indicators in the sensitive stream category, such as forest or crop cover.

1.2.2 Water Quality Response to Stormwater Pollution

As noted in the previous section, most ICM research has been done on streams, which are directly influenced by increased stormwater. Many managers have wondered whether the ICM also applies to downstream receiving waters, such as lakes, water supply reservoirs and small estuaries. In general, the exact water quality response of downstream receiving waters to increased nonpoint source pollutant loads depends on many factors, including the specific pollutant, the existing loading generated by the converted land use, and the geometry and hydraulics of the receiving water. Table 3 indicates the sensitivity of rivers, lakes, estuaries, aquifers and water supply reservoirs to various stormwater pollutants.

Lakes and the ICM

The water column and sediments of urban lakes are impacted by many stormwater pollutants, including sediment, nutrients, bacteria, metals, hydrocarbons, chlorides, and trash/debris. Of these pollutants, limnologists have always regarded phosphorus as the primary lake management concern, given that more than 80% of urban lakes experience symptoms of eutrophication (CWP, 2001a).

In general, phosphorus export steadily increases as IC is added to a lake watershed, although the precise amount of IC that triggers eutrophication problems is unique to each urban lake. With a little effort, it is possible to calculate the specific IC threshold for an individual lake, given its internal geometry, the size of its contributing watershed, current in-lake phosphorus concentration, degree of watershed treatment, and the desired water quality goals for the lake (CWP, 2001a). As a general rule, most lakes are extremely sensitive to increases in phosphorus loads caused by watershed IC. Exceptions include lakes that are unusually deep and/or have very small drainage area/lake area ratios. In most lakes, however, even a small amount of watershed development will result in an upward shift in trophic status (CWP, 2001a).

Reservoirs and the ICM

While surface water supply reservoirs respond to stormwater pollutant loads in the same general manner as lakes, they are subject to stricter standards because of their uses for drinking water. In particular, water supply reservoirs are particularly sensitive to increased turbidity, pathogens, total organic carbon, chlorides, metals, pesticides and hydrocarbon loads, in addition to phosphorus (Kitchell, 2001). While some pollutants can be removed or reduced through expanded filtering and treatment at drinking water intakes, the most reliable approach is to protect the source waters through watershed protection and treatment.

Consequently, we often recommend that the ICM be used as a “threat index” for most drinking water supplies. Quite simply, if current or future development is expected to exceed 10% IC in the contributing watershed, we recommend that a very aggressive watershed protection strategy be implemented (Kitchell, 2001). In addition, we contend that drinking water quality cannot be sustained once watershed IC exceeds 25% and have yet to find an actual watershed where a drinking water utility has been maintained under these conditions.

Small Tidal Estuaries and Coves and the ICM

The aquatic resources of small tidal estuaries, creeks, and coves are often highly impacted by watershed development and associated activities, such as boating/marinas, wastewater discharge, septic systems, alterations in freshwater flow and wetland degradation and loss. Given the unique impacts of eutrophication on the marine system and stringent water quality standards for shellfish harvesting, the stormwater pollutants of greatest concern in the estuarine water column are nitrogen and fecal coliform bacteria. Metals and hydrocarbons in stormwater runoff can also contaminate bottom sediments, which can prove toxic to local biota (Fortner *et al*., 1996; Fulton *et al.*, 1996; Kucklick *et al.*, 1997; Lerberg *et al.*, 2000; Sanger *et al.,* 1999; Vernberg *et al.*, 1992).

While numerous studies have demonstrated that physical, hydrologic, water quality and biological indicators differ in urban and non-urban coastal watersheds, only a handful of studies have used watershed IC as an indicator of estuarine health. These studies show significant correlations with IC, although degradation thresholds may not necessarily adhere to the ICM due to tidal dilution and dispersion. Given the limited research, it is not fully clear if the ICM can be applied to coastal systems without modification.

Atmospheric deposition is considered a primary source of nitrogen loading to estuarine watersheds. Consequently, nitrogen loads in urban stormwater are often directly linked to IC. Total nitrogen loads have also been linked to groundwater input, especially from subsurface discharges from septic systems, which are common in low density coastal development (Swann, 2001; Valiela *et al.*, 1997; Vernberg *et al.*, 1996a). Nitrogen is generally considered to be the limiting nutrient in estuarine systems, and increased loading has been shown to increase algal and phytoplankton biomass and cause shifts in the phytoplankton community and food web structure that may increase the potential for phytoplankton blooms and fish kills (Bowen and Valiela, 2001; Evgenidou *et al.*, 1997; Livingston, 1996).

Increased nitrogen loads have been linked to declining seagrass communities, finfish populations, zooplankton reproduction, invertebrate species richness, and shellfish populations (Bowen and Valiela, 2001; Rutkowski *et al.*, 1999; Short and Wyllie-Echeverria, 1996; Valiela and Costa, 1988). Multiple studies have shown significant increases in nitrogen loading as watershed land use becomes more urban (Valiela *et al.,* 1997; Vernberg *et al.* 1996a; Wahl *et al.,* 1997). While a few studies link nitrogen loads with building and population density, no study was found that used IC as an indicator of estuarine nitrogen loading.

The second key water quality concern in small estuaries is high fecal coliform levels in stormwater runoff, which can lead to the closure of shellfish beds and swimming beaches. Waterfowl and other wildlife have also been shown to contribute to fecal coliform loading (Wieskel *et al.*, 1996). Recent research has shown that fecal coliform standards are routinely violated during storm events at very low levels of IC in coastal watersheds (Mallin *et al.,* 2001; Vernberg *et al.*, 1996b; Schueler, 1999). Maiolo and Tschetter (1981) found a significant correlation between human population and closed shellfish acreage in North Carolina, and Duda and Cromartie (1982) found greater fecal coliform densities when septic tank density and IC increased, with an approximate threshold at 10% watershed IC.

Recently, Mallin *et al.* (2000) studied five small North Carolina estuaries of different land uses and showed that fecal coliform levels were significantly correlated with watershed population, developed land and IC. Percent IC was the most statistically significant indicator and could explain 95% of the variability in fecal coliform concentrations. They also found that shellfish bed closures were possible in watersheds with less than 10% IC, common in watersheds above 10% IC, and almost certain in watersheds above 20% IC. While higher fecal coliform levels were observed in developed watersheds, salinity, flushing and proximity to pollution sources often resulted in higher concentrations at upstream locations and at high tides (Mallin *et al.*, 1999). While these studies support the ICM, more research is needed to prove the reliability of the ICM in predicting shellfish bed closures based on IC.

Several studies have also investigated the impacts of urbanization on estuarine fish, macrobenthos and shellfish communities. Increased PAH accumulation in oysters, negative effects of growth in juvenile sheepshead minnows, reduced molting efficiency in copepods, and reduced numbers of grass shrimp have all been reported for urban estuaries as compared to forested estuaries (Fulton *et al.,* 1996). Holland *et al. (*1997) reported that the greatest abundance of penaid shrimp and mummichogs was observed in tidal creeks with forested watersheds compared to those with urban cover. Porter *et al.* (1997) found lower grass shrimp abundance in small tidal creeks adjacent to commercial and urban development, as compared to non-urban watersheds.

Lerberg *et al.* (2000) studied small tidal creeks and found that highly urban watersheds (50% IC) had the lowest benthic diversity and abundance as compared to suburban and forested creeks, and benthic communities were numerically dominated by tolerant oligochaetes and polychaetes. Suburban watersheds (15 to 35% IC) also showed signs of degradation and had some pollution tolerant macrobenthos, though not as markedly as urban creeks. Percent abundance of pollution-indicative species showed a marked decline at 30% IC, and the abundance of pollution-sensitive species also significantly correlated with IC (Lerberg *et al*., 2000). Holland *et al*. (1997) reported that the variety and food availability for juvenile fish species was impacted at 15 to 20% IC.

Lastly, a limited amount of research has focused on the direct impact of stormwater runoff on salinity and hypoxia in small tidal creeks. Blood and Smith (1996) compared urban and forested watersheds and found higher salinities in urban watersheds due to the increased number of impoundments. Fluctuations in salinity have been shown to affect shellfish and other aquatic populations (see Vernberg, 1996b). When urban and forested watersheds were compared, Lerberg *et al.* (2000) reported that higher salinity fluctuations occurred most often in developed watersheds; significant correlations with salinity range and IC were also determined. Lerberg *et al.* (2000) also found that the most severe and frequent hypoxia occurred in impacted salt marsh creeks and that dissolved oxygen dynamics in tidal creeks were comparable to dead-end canals common in residential marina-style coastal developments. Suburban watersheds (15 to 35% IC) exhibited signs of degradation and had some pollution-tolerant macrobenthic species, though not to the extent of urban watersheds (50% IC).

In summary, recent research suggests that indicators of coastal watershed health are linked to IC. However, more research is needed to clarify the relationship between IC and estuarine indicators in small tidal estuaries and high salinity creeks.

1.2.3 Effect of Watershed Treatment on Stormwater Quality

Over the past two decades, many communities have invested in watershed protection practices, such as stormwater treatment practices (STPs), stream buffers, and better site design, in order to reduce pollutant loads to receiving waters. In this section, we review the effect of watershed treatment on the quality of stormwater runoff.

Effect of Stormwater Treatment Practices

We cannot directly answer the question as to whether or not stormwater treatment practices can significantly reduce water quality impacts at the watershed level, simply because no controlled monitoring studies have yet been conducted at this scale. Instead, we must rely on more indirect research that has tracked the change in mass or concentration of pollutants as they travel through individual stormwater treatment practices. Thankfully, we have an abundance of these performance studies, with nearly 140 monitoring studies evaluating a diverse range of STPs, including ponds, wetlands, filters, and swales (Winer, 2000).

These studies have generally shown that stormwater practices have at least a moderate ability to remove many pollutants in urban stormwater. Table 4 provides average removal efficiency rates for a range of practices and stormwater pollutants, and Table 5 profiles the mean storm outflow concentrations for various practices. As can be seen, some groups of practices perform better than others in removing certain stormwater pollutants. Consequently, managers need to carefully choose which practices to apply to solve the primary water quality problems within their watersheds.

It is also important to keep in mind that site-based removal rates cannot be extrapolated to the watershed level without significant adjustment. Individual site practices are never implemented perfectly or consistently across a watershed. At least three discount factors need to be considered: bypassed load, treatability and loss of performance over time. For a review on how these discounts are derived, consult Schueler and Caraco (2001). Even under the most optimistic watershed implementation scenarios, overall pollutant reduc-

Table 4: The Effectiveness of Stormwater Treatment Practices in Removing Pollutants - Percent Removal Rate (Winer, 2000)

Practice	N	TSS	TP	OP	TN	NOx	Cu	Zn	Oil/ Grease[1]	Bacteria
Dry Ponds	9	47	19	N/R	25	3.5	26	26	3	44
Wet Ponds	43	80	51	65	33	43	57	66	78	70
Wetlands	36	76	49	48	30	67	40	44	85	78
Filtering Practices[2]	18	86	59	57	38	-14	49	88	84	37
Water Quality Swales	9	81	34	1.0	84	31	51	71	62	-25
Ditches[3]	9	31	-16	N/R	-9.0	24	14	0	N/R	0
Infiltration	6	95	80	85	51	82	N/R	N/R	N/R	N/R

1: Represents data for Oil and Grease and PAH
2: Excludes vertical sand filters
3: Refers to open channel practices not designed for water quality
N/R = Not Reported

Table 5: Median Effluent Concentrations from Stormwater Treatment Practices (mg/l) (Winer, 2000)								
Practice	N	TSS	TP	OP	TN	NOx	Cu[1]	Zn[1]
Dry Ponds[2]	3	28	0.18	N/R	0.86	N/R	9.0	98
Wet Ponds	25	17	0.11	0.03	1.3	0.26	5.0	30
Wetlands	19	22	0.20	0.07	1.7	0.36	7.0	31
Filtering Practices[3]	8	11	0.10	0.07	1.1	0.55	9.7	21
Water Quality Swales	7	14	0.19	0.09	1.1	0.35	10	53
Ditches[4]	3	29	0.31	N/R	2.4	0.72	18	32

1. Units for Zn and Cu are micrograms per liter (Fg/l)
2. Data available for Dry Extended Detention Ponds only
3. Excludes vertical sand filters
4. Refers to open channel practices not designed for water quality
N/R = Not Reported

tions by STPs may need to be discounted by at least 30% to account for partial watershed treatment.

Even with discounting, however, it is evident that STPs can achieve enough pollutant reduction to mimic rural background loads for many pollutants, as long as the watershed IC does not exceed 30 to 35%. This capability is illustrated in Figure 5, which shows phosphorus load as a function of IC, with and without stormwater treatment.

Effect of Stream Buffers/Riparian Areas

Forested stream buffers are thought to have very limited capability to remove stormwater pollutants, although virtually no systematic monitoring data exists to test this hypothesis. The major reason cited for their limited removal capacity is that stormwater generated from upland IC has usually concentrated before it reaches the forest buffer and therefore crosses the buffer in a channel, ditch or storm drain pipe. Consequently, the opportunity to filter runoff is lost in many forest buffers in urban watersheds.

Effect of Better Site Design

Better site design (BSD) is a term for nonstructural practices that minimize IC, conserve natural areas and distribute stormwater treatment across individual development sites. BSD is also known by many other names, including conservation development, low-impact development, green infrastructure, and sustainable urban drainage systems. While

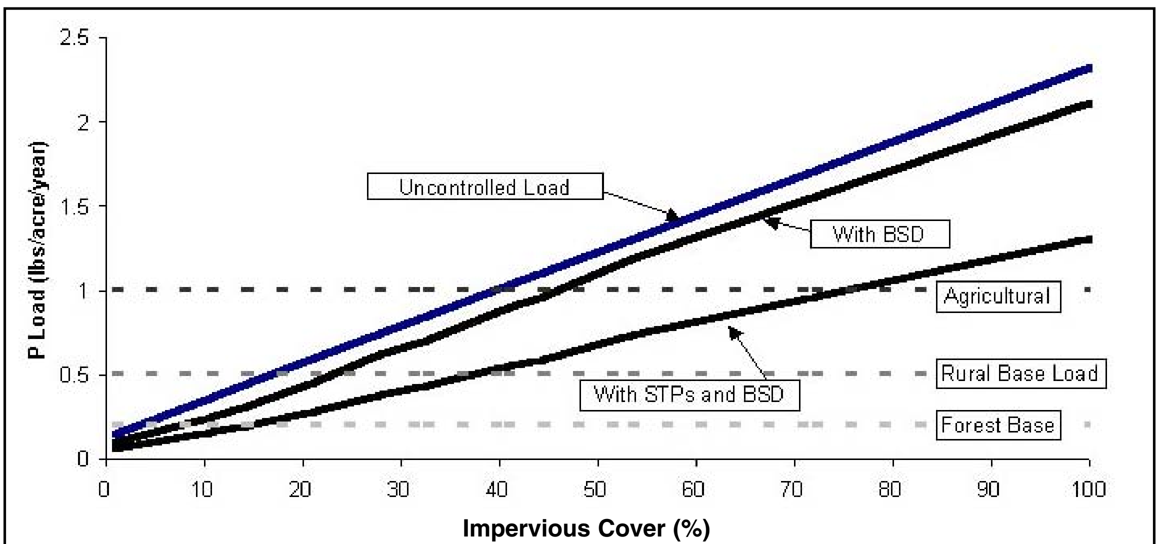


Figure 5: Estimated Phosphorus Load as a Function of Impervious Cover, Discounted Stormwater Treatment and Better Site Design (Schueler and Caraco, 2001)

some maintain that BSD is an alternative to traditional STPs, most consider it to be an important complement to reduce pollutant loads.

While BSD has become popular in recent years, only one controlled research study has evaluated its potential performance, and this is not yet complete (i.e. Jordan Cove, CT).

Indirect estimates of the potential value of BSD to reduce pollutant discharges have been inferred from modeling and redesign analyses (Zielinski, 2000). A typical example is provided in Figure 5, which shows the presumed impact of BSD in reducing phosphorus loadings. As is apparent, BSD appears to be a very effective strategy in the one to 25% IC range, but its benefits diminish beyond that point.

1.3 Implications of the ICM for Watershed Managers

One of the major policy implications of the ICM is that in the absence of watershed treatment, it predicts negative stream impacts at an extremely low intensity of watershed development. To put this in perspective, consider that a watershed zoned for two-acre lot residential development will generally exceed 10% IC, and therefore shift from a sensitive to an impacted stream classification (Cappiella and Brown, 2001). Thus, if a community wants to protect an important water resource or a highly regarded species (such as trout, salmon or an endangered freshwater mussel), the ICM suggests that there is a maximum limit to growth that is not only quite low, but is usually well below the current zoning for many suburban or even rural watersheds. Consequently, the ICM suggests the unpleasant prospect that massive down-zoning, with all of the associated political and legal carnage involving property rights and economic development, may be required to maintain stream quality.

It is not surprising, then, that the ICM debate has quickly shifted to the issue of whether or not watershed treatment practices can provide adequate mitigation for IC. How much relief can be expected from stream buffers, stormwater ponds, and other watershed practices, which might allow greater development density within a given watershed? Only a limited amount of research has addressed this question, and the early results are not reassuring (reviewed in section 1.1.3). At this early stage, researchers are still having trouble detecting the impact of watershed treatment, much less defining it. As noted earlier, both watershed research techniques and practice implementation need to be greatly improved if we ever expect to get a scientifically defensible answer to this crucial question. Until then, managers should be extremely cautious in setting high expectations for how much watershed treatment can mitigate IC.

1.3.1 Management of Non-Supporting Streams

Most researchers acknowledge that streams with more than 25% IC in their watersheds cannot support their designated uses or attain water quality standards and are severely degraded from a physical and biological standpoint. As a consequence, many of these streams are listed for non-attainment under the Clean Water Act and are subject to Total Maximum Daily Load (TMDL) regulations. Communities that have streams within this regulatory class must prepare implementation plans that demonstrate that water quality standards can ultimately be met.

While some communities have started to restore or rehabilitate these streams in recent years, their efforts have yielded only modest improvements in water quality and biological indicators. In particular, no community has yet demonstrated that they can achieve water quality standards in an urban watershed that exceeds 25% IC. Many communities are deeply concerned that non-supporting streams may never achieve water quality standards, despite massive investments in watershed restoration. The ICM suggests that water quality standards may need to be sharply revised for streams with more than 25% IC, if they are ever to come into attainment. While states have authority to create more achievable standards for non-supporting streams within the regulatory framework of the Clean Water Act (Swietlik, 2001), no state has yet exercised this authority. At this time, we are not aware of any water quality standards that are based on the ICM or similar urban stream classification techniques.

Two political perceptions largely explain why states are so reticent about revising water quality standards. The first is a concern that they will run afoul of anti-degradation provisions within the Clean Water Act or be accused of "backsliding" by the environmental community. The second concern relates to the demographics of watershed organizations across the country. According to recent surveys, slightly more than half of all watershed organizations represent moderately to highly developed watersheds (CWP, 2001a). These urban watershed organizations often have a keen interest in keeping the existing regulatory structure intact, since it is perceived to be the only lever to motivate municipalities to implement restoration efforts in non-supporting streams.

However, revised water quality standards are urgently needed to support smart growth efforts. A key premise of smart growth is that it is more desirable to locate new development within a non-supporting subwatershed rather than a sensitive or impacted one (i.e., concentrating density and IC within an existing subwatershed helps prevent sprawl from encroaching on a less developed one). Yet while smart growth is desirable on a regional basis, it will usually contribute to already serious problems in non-supporting watersheds, which makes it even more difficult to meet water quality standards.

This creates a tough choice for regulators: if they adopt stringent development criteria for non-supporting watersheds, their added costs can quickly become a powerful barrier to desired redevelopment. If, on the other hand, they relax or waive environmental criteria, they contribute to the further degradation of the watershed. To address this problem, the Center has developed a "smart watersheds" program to ensure that any localized degradation caused by development within a non-supporting subwatershed is more than compensated for by improvements in stream quality achieved through municipal restoration efforts (CWP, in press). Specifically, the smart watersheds program includes 17 public sector programs to treat stormwater runoff, restore urban stream corridors and reduce pollution discharges in highly urban watersheds. It is hoped that communities that adopt and implement smart watershed programs will be given greater flexibility to meet state and federal water quality regulations and standards within non-supporting watersheds.

1.3.2 Use of the ICM for Urban Stream Classification

The ICM has proven to be a useful tool for classifying and managing the large inventory of streams that most communities possess. It is not unusual for a typical county to have several thousand miles of headwater streams within its political boundaries, and the ICM provides a unified framework to identify and manage these subwatersheds. In our watershed practice, we use the ICM to make an initial diagnosis rather than a final determination for stream classification. Where possible, we conduct rapid stream and subwatershed assessments as a final check for an individual stream classification, particularly if it borders between the sensitive and impacted category. As noted earlier, the statistical variation in the IC/stream quality indicator makes it difficult to distinguish between a stream with 9% versus 11% IC. Some of the key criteria we use to make a final stream classification are provided in Table 6.

1.3.3 Role of the ICM in Small Watershed Planning

The ICM has also proven to be an extremely important tool for watershed planning, since it can rapidly project how streams will change in response to future land use. We routinely estimate existing and future IC in our watershed planning practice and find that it is an excellent indicator of change for subwatersheds in the zero to 30% IC range. In particular, the ICM often forces watershed planners to directly confront land use planning and land conservation issues early in the planning process.

On the other hand, we often find that the ICM has limited planning value when subwatersheds exceed 30% IC for two practical reasons. First, the ICM does not differentiate stream conditions within this very large span of IC (i.e., there is no difference in the stream quality prediction for a subwatershed that has 39.6% IC versus one that has 58.4% IC). Second, the key management question for non-supporting watersheds is whether or not they are potentially restorable. More detailed analysis and field investigations are needed to determine, in each subwatershed, the answer to this question. While a knowledge of IC is often used in these feasibility assessments, it is but one of many factors that needs to be considered.

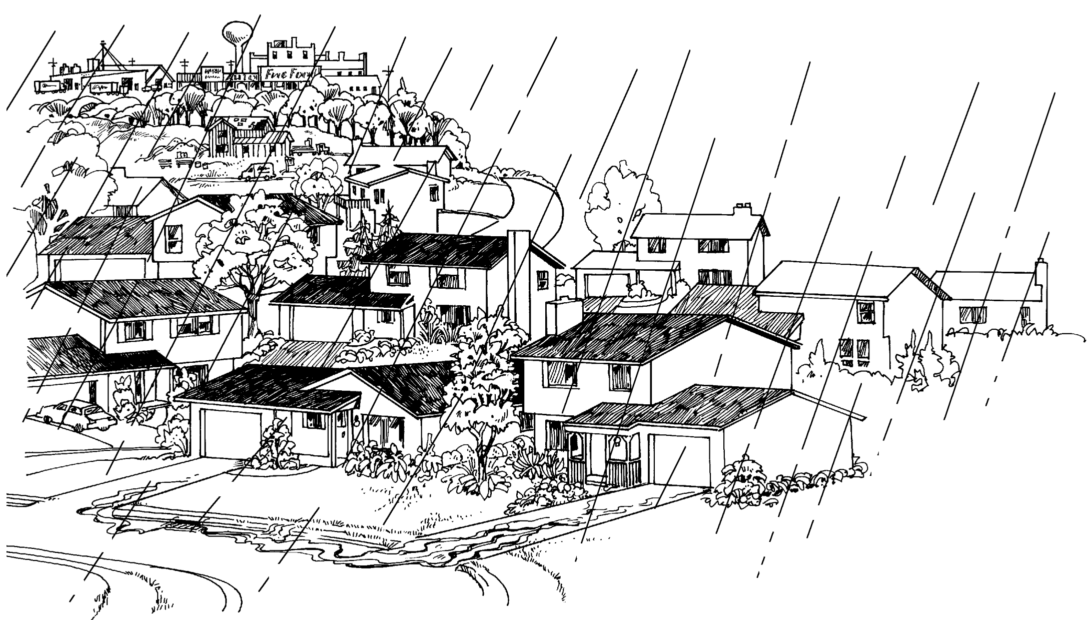
Lastly, we have come to recognize several practical factors when applying the ICM for small watershed planning. These include thoughtful delineation of subwatershed boundaries, the proper accounting of a direct drainage area in larger watersheds, and the critical need for the most recent IC data. More guidance on these factors can be found in Zielinski (2001).

Impervious cover is not a perfect indicator of existing stream quality. A number of stream and subwatershed criteria should be evaluated in the field before a final classification decision is made, particularly when the stream is on the borderline between two classifications. We routinely look at the stream and subwatershed criteria to decide whether a borderline stream should be classified as sensitive or impacted. Table 6 reviews these additional criteria.

Table 6: Additional Considerations for Urban Stream Classification
Stream Criteria
Reported presence of rare, threatened or endangered species in the aquatic community (e.g., freshwater mussels, fish, crayfish or amphibians) Confirmed spawning of cold-water fish species (e.g., trout) Fair/good, good, or good to excellent macro invertebrate scores More than 65% of EPT species present in macro-invertebrate surveys No barriers impede movement of fish between the subwatershed and downstream receiving waters Stream channels show little evidence of ditching, enclosure, tile drainage or channelization Water quality monitoring indicates no standards violations during dry weather Stream and flood plain remain connected and regularly interact Stream drains to a downstream surface water supply Stream channels are generally stable, as determined by the Rosgen level analysis Stream habitat scores are rated at least fair to good
Subwatershed Criteria
Contains terrestrial species that are documented as rare, threatened and endangered Wetlands, flood plains and/or beaver complexes make up more than 10% of subwatershed area Inventoried conservation areas comprise more than 10% of subwatershed area More than 50% of the riparian forest corridor has forest cover and is either publicly owned or regulated Large contiguous forest tracts remain in the subwatershed (more than 40% in forest cover) Significant fraction of subwatershed is in public ownership and management Subwatershed connected to the watershed through a wide corridor Farming, ranching and livestock operations in the subwatershed utilize best management practices Prior development in the subwatershed has utilized stormwater treatment practices

1.4 Summary

The remainder of this report presents greater detail on the individual research studies that bear on the ICM. Chapter 2 profiles research on hydrologic indicators in urban streams, while Chapter 3 summarizes the status of current research on the impact of urbanization on physical habitat indicators. Chapter 4 presents a comprehensive review of the impact of urbanization on ten major stormwater pollutants. Finally, Chapter 5 reviews the growing body of research on the link between IC and biological indicators within urban streams and wetlands.



Chapter 2: Hydrologic Impacts of Impervious Cover

The natural hydrology of streams is fundamentally changed by increased watershed development. This chapter reviews the impacts of watershed development on selected indicators of stream hydrology.

This chapter is organized as follows:

2.1 Introduction
2.2 Increased Runoff Volume
2.3 Increased Peak Discharge Rates
2.4 Increased Bankfull Flow
2.5 Decreased Baseflow
2.6 Conclusions

2.1 Introduction

Fundamental changes in urban stream hydrology occur as a result of three changes in the urban landscape that accompany land development. First, large areas of the watershed are paved, rendering them impervious. Second, soils are compacted during construction, which significantly reduces their infiltration capabilities. Lastly, urban stormwater drainage systems are installed that increase the efficiency with which runoff is delivered to the stream (i.e., curbs and gutters, and storm drain pipes). Consequently, a greater fraction of annual rainfall is converted to surface runoff, runoff occurs more quickly, and peak flows become larger. Additionally, dry weather flow in streams may actually decrease because less groundwater recharge is available. Figure 6 illustrates the change in hydrology due to increased urban runoff as compared to pre-development conditions.

Research has demonstrated that the effect of watershed urbanization on peak discharge is more marked for smaller storm events. In particular, the bankfull, or channel forming flow, is increased in magnitude, frequency and duration. Increased bankfull flows have strong ramifications for sediment transport and channel enlargement. All of these changes in the natural water balance have impacts on the physical structure of streams, and ultimately affect water quality and biological diversity.

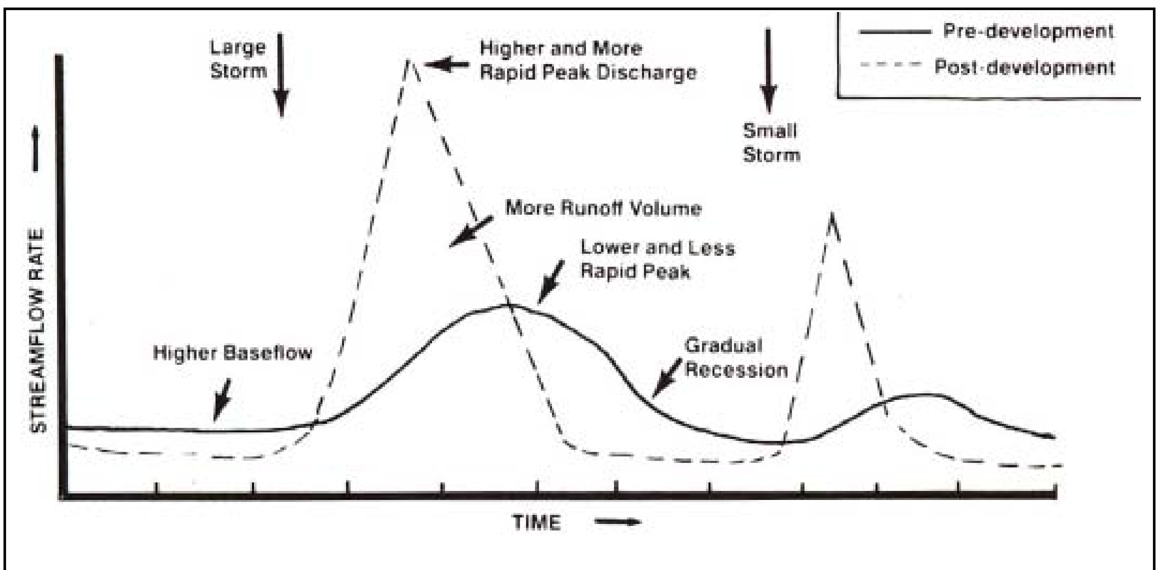
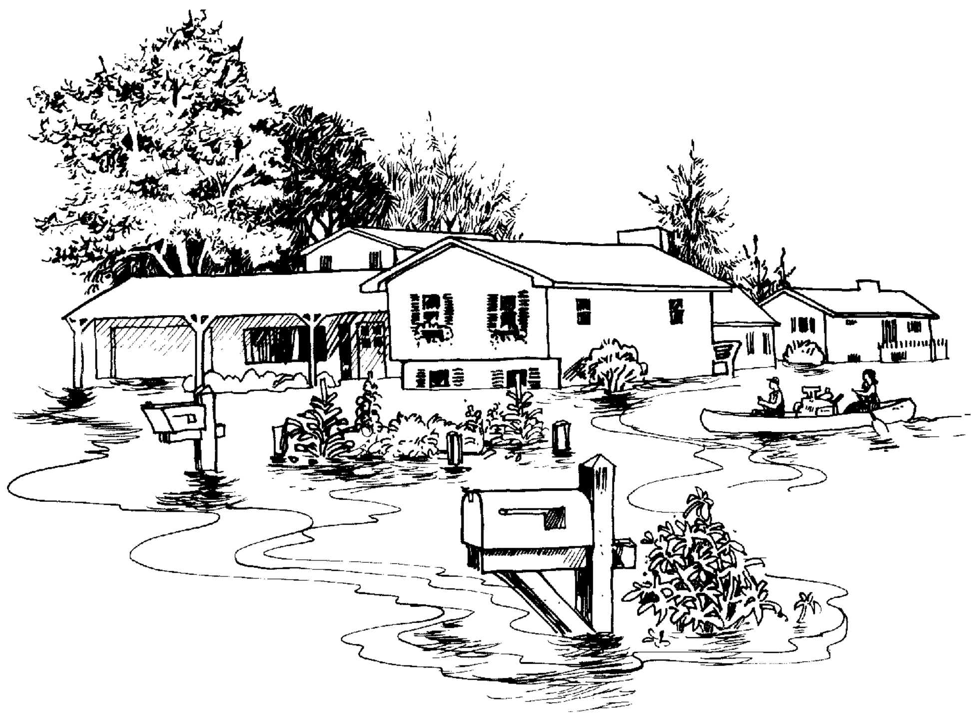


Figure 6: Altered Hydrograph in Response to Urbanization (Schueler, 1987)

The relationship between watershed IC and stream hydrology is widely accepted, and has been incorporated into many hydrologic engineering models over the past three decades. Several articles provide a good summary of these (Bicknell *et al.*, 1993; Hirsch *et al.*, 1990; HEC, 1977; Huber and Dickinson, 1988; McCuen and Moglen, 1988; Overton and Meadows, 1976; Pitt and Voorhees, 1989; Schueler, 1987; USDA, 1992; 1986).

The primary impacts of watershed development on stream hydrology are as follows:

- Increased runoff volume
- Increased peak discharge rates
- Increased magnitude, frequency, and duration of bankfull flows
- Diminished baseflow

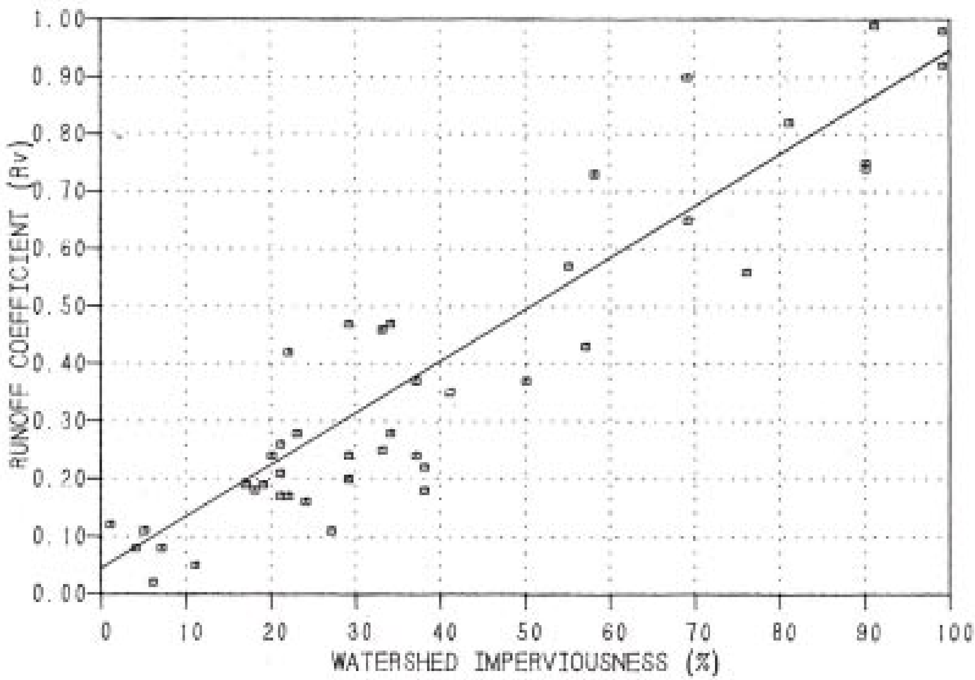


2.2 Increased Runoff Volume

Impervious cover and other urban land use alterations, such as soil compaction and storm drain construction, alter infiltration rates and increase runoff velocities and the efficiency with which water is delivered to streams. This decrease in infiltration and basin lag time can significantly increase runoff volumes. Table 7 reviews research on the impact of IC on runoff volume in urban streams. Schueler (1987) demonstrated that runoff values are directly related to subwatershed IC (Figure 7). Runoff data was derived from 44 small catchment areas across the country for EPA's Nationwide Urban Runoff Program.

Table 8 illustrates the difference in runoff volume between a meadow and a parking lot, as compiled from engineering models. The parking lot produces more than 15 times more runoff than a meadow for the same storm event.

Urban soils are also profoundly modified during the construction process. The compaction of urban soils and the removal of topsoil can decrease the infiltration capacity, causing increases in runoff volumes (Schueler, 2000). Bulk density is often used to measure soil compaction, and Table 9 illustrates how bulk density increases in many urban land uses.



Note: 44 small urban catchments monitored during the national NURP study

Figure 7: Runoff Coefficient vs. IC (Schueler, 1987)

Table 7: Research Review of Increased Runoff Volume and Peak Discharge in Urban Streams		
Reference	**Key Finding**	**Location**
	Increased Runoff Volume	
Schueler, 1987	Runoff coefficients were found to be strongly correlated with IC at 44 sites nationwide.	U.S.
Neller, 1988	Urban watershed produced more than seven times as much runoff as a similar rural watershed. Average time to produce runoff was reduced by 63% in urban watersheds compared to rural watersheds.	Australia
	Increased Peak Discharge	
Hollis, 1975	Review of data from several studies showed that floods with a return period of a year or longer are not affected by a 5% watershed IC; small floods may be increased 10 times by urbanization; flood with a return period of 100 years may be doubled in size by a 30% watershed IC.	N/A
Leopold, 1968	Data from seven nationwide studies showed that 20% IC can cause the mean annual flood to double.	U.S.
Neller, 1988	Average peak discharge from urban watersheds was 3.5 times higher than peak runoff from rural watersheds.	Australia
Doll *et al.*, 2000	Peak discharge was greater for 18 urban streams versus 11 rural Piedmont streams.	NC
Sauer *et al.*, 1983	Estimates of flood discharge for various recurrence intervals showed that less than 50% watershed IC can result in a doubling of the 2-year, 10-year, and 100-year floods.	U.S.
Leopold, 1994	Watershed development over a 29-year period caused the peak discharge of the 10-year storm to more than double.	MD
Kibler *et al.*, 1981	Rainfall/runoff model for two watersheds showed that an increase in IC caused a significant increase in mean annual flood.	PA
Konrad and Booth, 2002	Evaluated streamflow data at 11 streams and found that the fraction of annual mean discharges was exceeded and maximum annual instantaneous discharges were related to watershed development and road density for moderately and highly developed watersheds.	WA

Table 8: Hydrologic Differences Between a Parking Lot and a Meadow (Schueler, 1994a)		
Hydrologic or Water Quality Parameter	**Parking Lot**	**Meadow**
Runoff Coefficient	0.95	0.06
Time of Concentration (minutes)	4.8	14.4
Peak Discharge, two-year, 24-hour storm (cfs)	4.3	0.4
Peak Discharge Rate, 100-year storm (cfs)	12.6	3.1
Runoff Volume from one-inch storm (cu. ft)	3,450	218
Runoff Velocity @ two-year storm (ft/sec)	8	1.8

Key Assumptions:
2-yr, 24-hr storm = 3.1 in; 100-yr storm = 8.9 in.
Parking Lot: 100% imperviousness; 3% slope; 200ft flow length; hydraulic radius =.03; concrete channel; suburban Washington C values
Meadow: 1% impervious; 3% slope; 200 ft flow length; good vegetative condition; B soils; earthen channel
Source: Schueler, 1994a

Table 9: Comparison of Bulk Density for Undisturbed Soils and Common Urban Conditions (Schueler, 2000)			
Undisturbed Soil Type or Urban Condition	**Surface Bulk Density (grams/cubic centimeter)**	**Urban Condition**	**Surface Bulk Density (grams/cubic centimeter)**
Peat	0.2 to 0.3	Urban Lawns	1.5 to 1.9
Compost	1.0	Crushed Rock Parking Lot	1.5 to 1.9
Sandy Soils	1.1 to 1.3	Urban Fill Soils	1.8 to 2.0
Silty Sands	1.4	Athletic Fields	1.8 to 2.0
Silt	1.3 to 1.4	Rights-of-Way and Building Pads (85%)	1.5 to 1.8
Silt Loams	1.2 to 1.5	Rights-of-Way and Building Pads (95%)	1.6 to 2.1
Organic Silts/Clays	1.0 to 1.2	Concrete Pavement	2.2
Glacial Till	1.6 to 2.0	Rock	2.65

2.3 Increased Peak Discharge Rate

Watershed development has a strong influence on the magnitude and frequency of flooding in urban streams. Peak discharge rates are often used to define flooding risk. Doll *et al.* (2000) compared 18 urban streams with 11 rural streams in the North Carolina Piedmont and found that unit area peak discharge was always greater in urban streams (Figure 8). Data from Seneca Creek, Maryland also suggest a similar increase in peak discharge. The watershed experienced significant growth during the 1950s and 1960s. Comparison of pre- and post-development gage records suggests that the peak 10-year flow event more than doubled over that time (Leopold, 1994).

Hollis (1975) reviewed numerous studies on the effects of urbanization on floods of different recurrence intervals and found that the effect of urbanization diminishes when flood recurrence gets longer (i.e., 50 and 100 years). Figure 9 shows the effect on flood magnitude in urban watersheds with 30% IC, and shows the one-year peak discharge rate increasing by a factor of 10, compared to an undeveloped watershed. In contrast, floods with a 100-year recurrence interval only double in size under the same watershed conditions.

Sauer *et al.* (1983) evaluated the magnitude of flooding in urban watersheds throughout the United States. An equation was developed for estimating discharge for floods of two-year, 10-year, and 100-year recurrence intervals. The equations used IC to account for increased runoff volume and a basin development factor to account for sewers, curbs and gutters, channel improvements and drainage development. Sauer noted that IC is not the dominant factor in determining peak discharge rates for extreme floods because these storm events saturate the soils of undeveloped watersheds and produce high peak discharge rates. Sauer found that watersheds with 50% IC can increase peak discharge for the two-year flood by a factor of four, the 10-year flood by a factor of three, and the 100-year flood by a factor of 2.5, depending on the basin development factor (Figure 10).

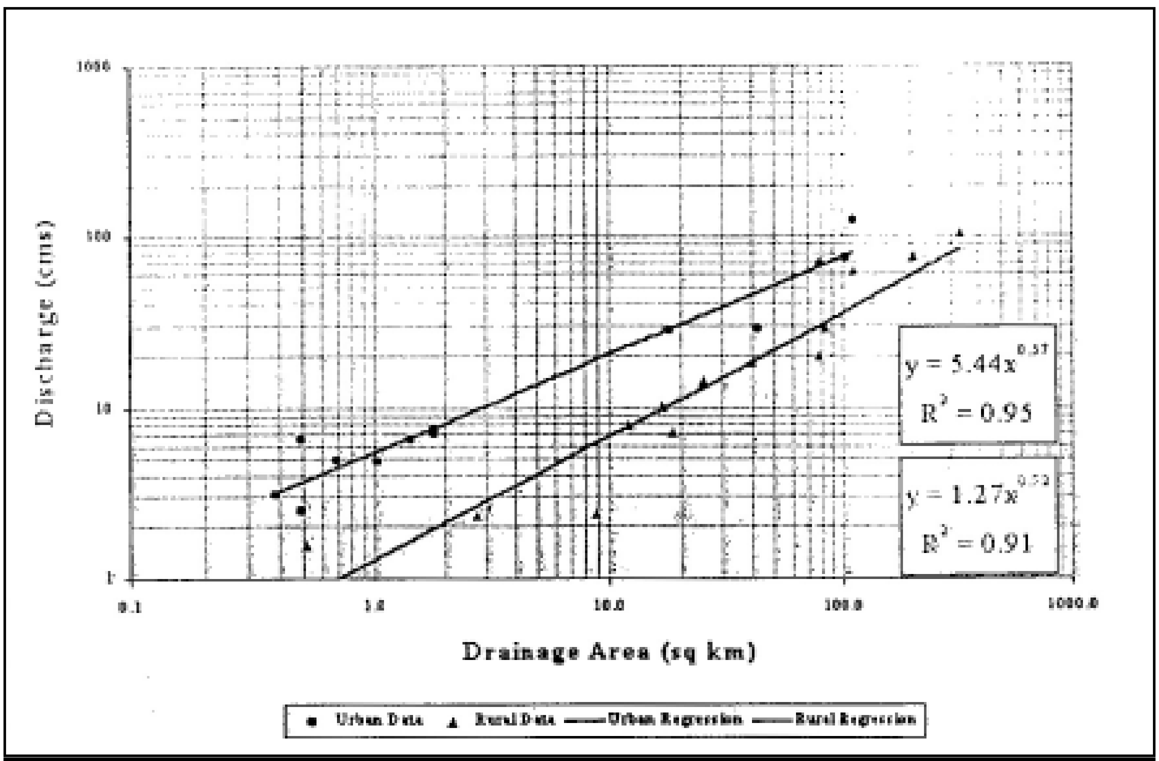


Figure 8: Peak Discharge for Urban and Rural Streams in North Carolina (Doll *et al.*, 2000)

2.4 Increased Bankfull Flow

Urbanization also increases the frequency and duration of peak discharge associated with smaller flood events (i.e., one- to two-year return storms). In terms of stream channel morphology, these more frequent bankfull flows are actually much more important than large flood events in forming the channel. In fact, Hollis (1975) demonstrated that urbanization increased the frequency and magnitude of bankfull flow events to a greater degree than the larger flood events.

An example of the increase in bankfull flow in arid regions is presented by the U.S. Geological Survey (1996), which compared the peak discharge rate from two-year storm events before and after watersheds urbanized in Parris Valley, California. Over an approximately 20-year period, watershed IC increased by 13.5%, which caused the two-year peak flow to more than double. Table 10 reviews other research studies on the relationship between watershed IC and bankfull flows in urban streams.

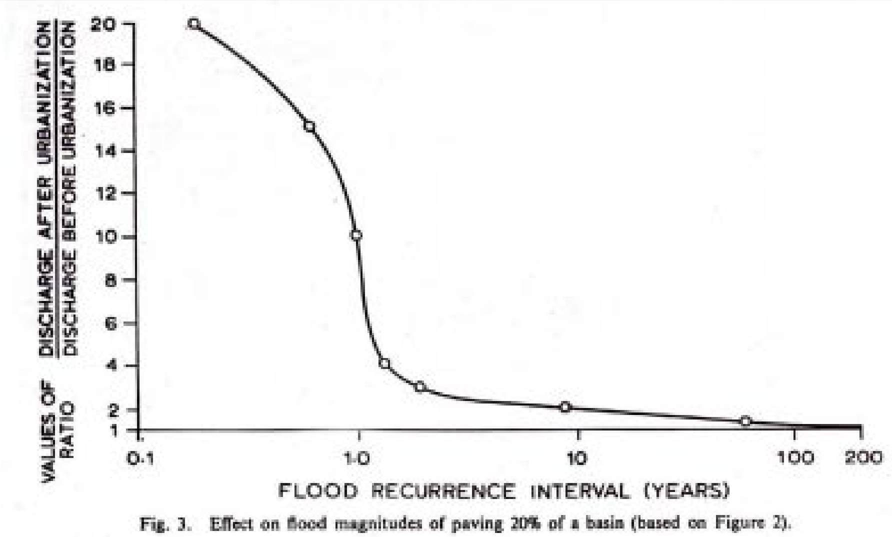


Figure 9: Effect on Flood Magnitudes of 30% Basin IC (Hollis, 1975)

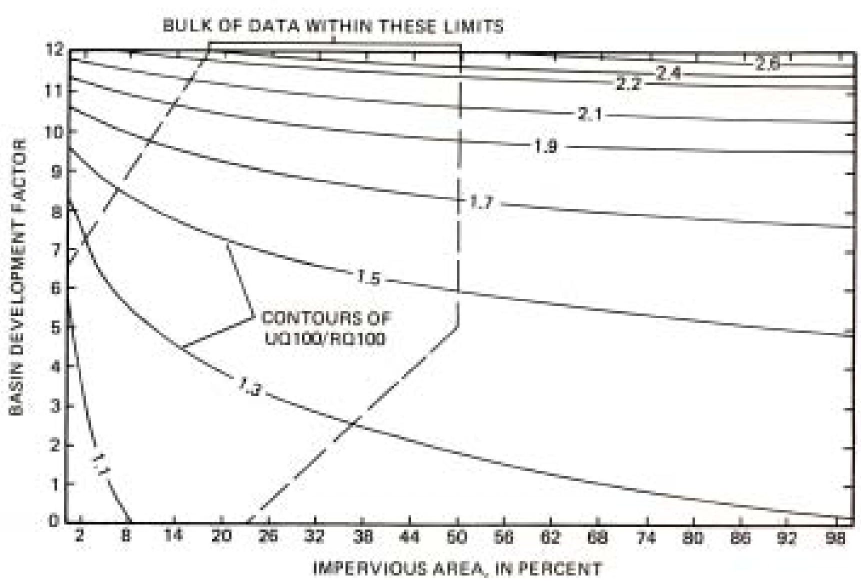


Figure 10: Relationship of Urban/Rural 100-Year Peak Flow Ratio to Basin Development Factor and IC (Sauer *et al.*, 1983)

Table 10: Research Review of Increased Bankfull Discharge in Urban Streams		
Reference	Key Finding	Location
Booth and Reinelt, 1993	Using a simulation model and hydrologic data from four watersheds, it was estimated that more than 10% watershed IC may cause discharge from the two-year storm under current conditions to equal or exceed discharge from the 10-year storm under forested conditions.	WA
Fongers and Fulcher, 2001	Bankfull flow of 1200 cfs was exceeded more frequently over time with urbanization, and exceedence was three times as frequent from 1930s to 1990s.	MI
USGS, 1996	Over a 20-year period, IC increased 13.5%, and the two-year peak flow more than doubled in a semi-arid watershed.	CA
Henshaw and Booth, 2000	Two of three watersheds in the Puget Sound lowlands showed increasing flashiness over 50 years with urbanization.	WA
Leopold, 1968	Using hydrologic data from a nine-year period for North Branch Brandywine Creek, it was estimated that for a 50% IC watershed, bankfull frequency would be increased fourfold.	PA
Leopold, 1994	Bankfull frequency increased two to seven times after urbanization in Watts Branch.	MD
MacRae, 1996	For a site downstream of a stormwater pond in Markham, Ontario hours of exceedence of bankfull flows increased by 4.2 times after the watershed urbanized (34% IC)	Ontario

Leopold (1968) evaluated data from seven nationwide studies and extrapolated this data to illustrate the increase in bankfull flows due to urbanization. Figure 11 summarizes the relationship between bankfull flows over a range of watershed IC. For example, watersheds that have 20% IC increase the number of flows equal to or greater than bankfull flow by a factor of two. Leopold (1994) also observed a dramatic increase in the frequency of the bankfull event in Watts Branch, an urban subwatershed in Rockville, Maryland. This watershed experienced significant urban development during the 1950s and 1960s. Leopold compared gage records and found that the bankfull storm event frequency increased from two to seven times per year from 1958 to 1987.

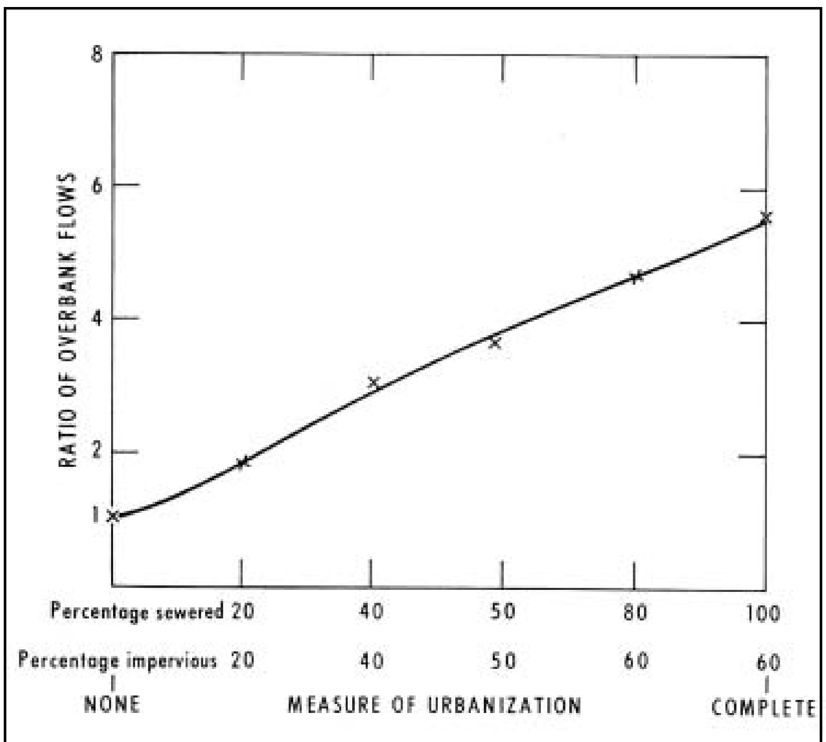


Figure 11: Increase in Bankfull Flows Due to Urbanization (Leopold, 1968)

More recent data on bankfull flow frequency was reported for the Rouge River near Detroit, Michigan by Fongers and Fulcher (2001). They noted that channel-forming flow (1200 cfs) was exceeded more frequently as urbanization increased in the watershed and had become three times more frequent between 1930 and 1990 (Figure 12).

McCuen and Moglen (1988) have documented the increase in duration of bankfull flows in response to urbanization using hydrology models. MacRae (1996), monitored a stream in Markham, Ontario downstream of a stormwater pond and found that the hours of

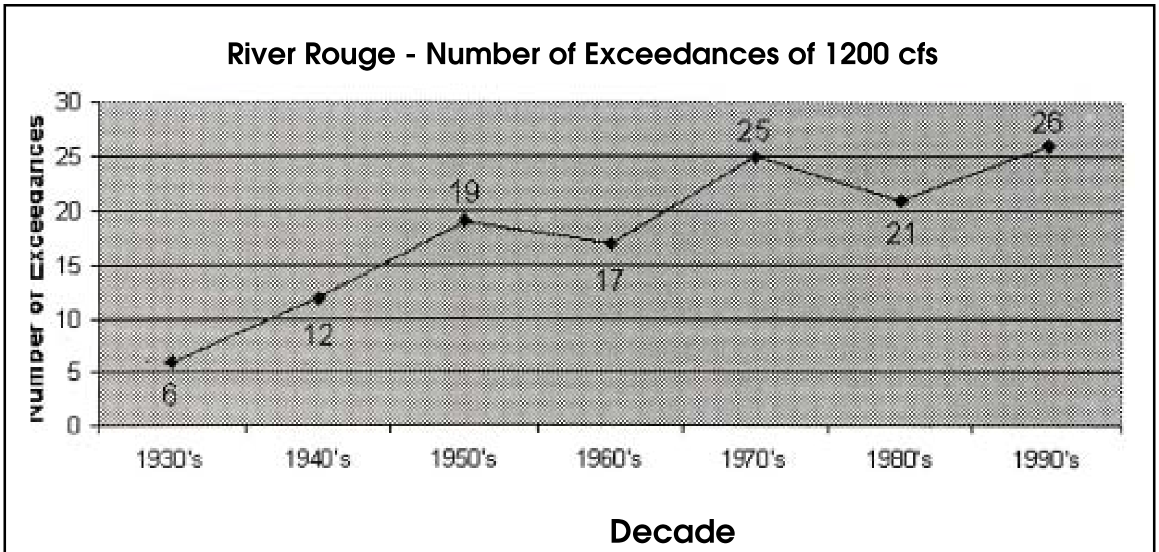


Figure 12: Increase in Number of Exceedences of Bankfull Flow Over Time With Urbanization in the Rouge River, MT (Fongers and Fulcher, 2001)

exceedence of bankfull flows increased by a factor of 4.2 once watershed IC exceeded 30%. Modeling for seven streams also downstream of stormwater ponds in Surrey, British Columbia also indicated an increase in bankfull flooding in response to watershed development (MacRae, 1996).

Watershed IC also increases the "flashiness" of stream hydrographs. Flashiness is defined here as the percent of daily flows each year that exceeds the mean annual flow. Henshaw and Booth (2000) evaluated seven urbanized watersheds in the Puget Sound lowland streams and tracked changes in flashiness over 50 years (Figure 13). The most urbanized watersheds experienced flashy discharges. Henshaw and Booth concluded that increased runoff in urban watersheds leads to higher but shorter-duration peak discharges.

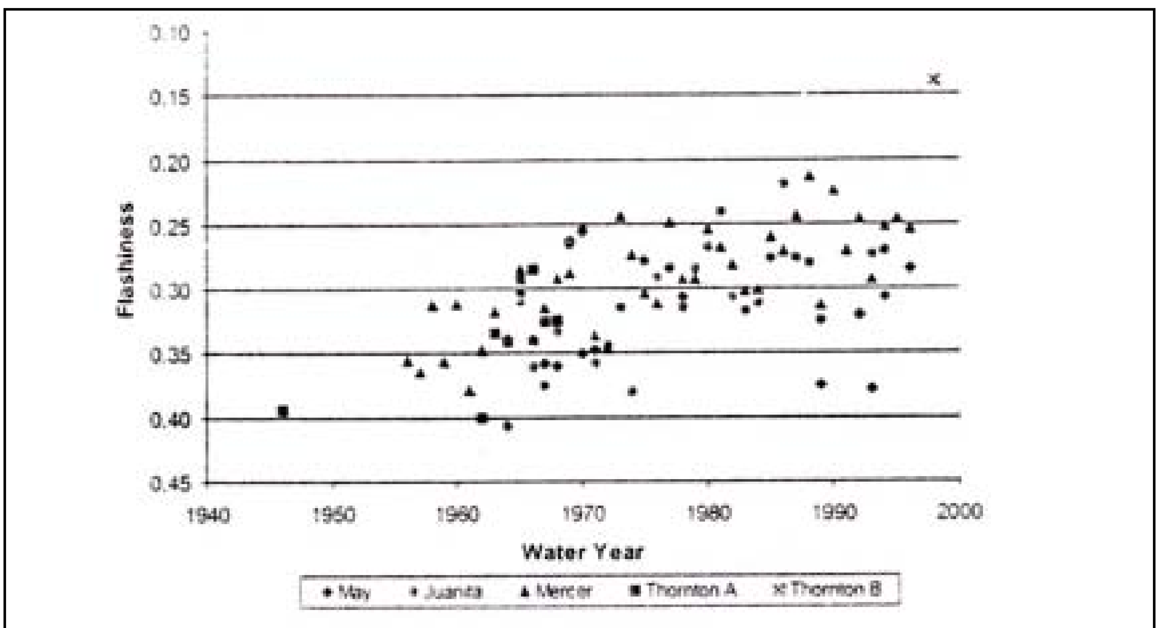


Figure 13: Percent of Gage Reading Above Mean Annual Flow for Puget Sound Lowland Streams (Henshaw and Booth, 2000)

2.5 Decreased Baseflow

As IC increases in a watershed, less groundwater infiltration is expected, which can potentially decrease stream flow during dry periods, (i.e. baseflow). Several East Coast studies provide support for a decrease in baseflow as a result of watershed development. Table 11 reviews eight research studies on baseflow in urban streams.

Klein (1979) measured baseflow in 27 small watersheds in the Maryland Piedmont and reported an inverse relationship between IC and baseflow (Figure 14). Spinello and Simmons (1992) demonstrated that baseflow in two urban Long Island streams declined seasonally as a result of urbanization (Figure 15). Saravanapavan (2002) also found that percentage of baseflow decreased in direct proportion to percent IC for 13 subwatersheds of the Shawsheen River watershed in Massachusetts (Figure 16).

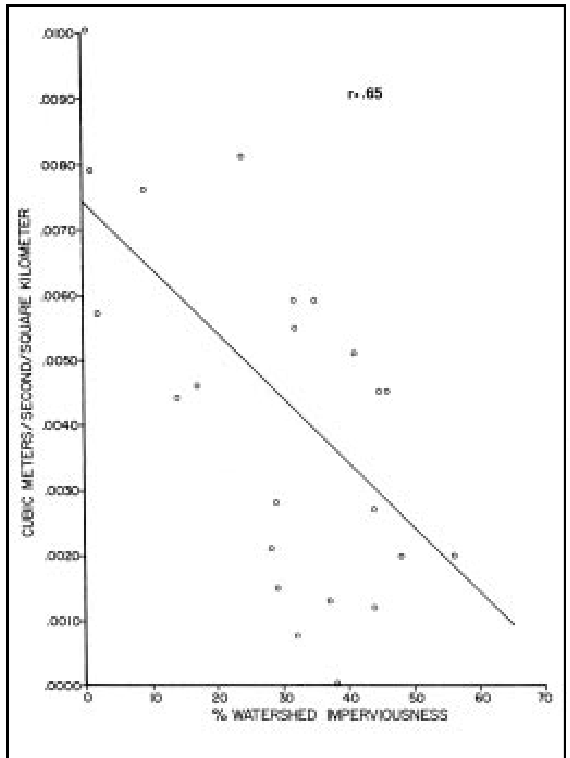
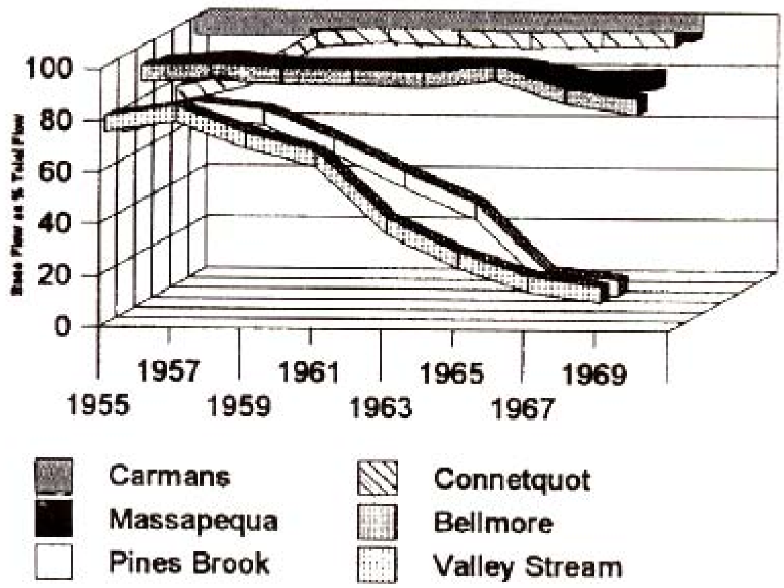


Figure 14: Relationship Between Baseflow and Watershed IC in the Streams on Maryland Piedmont (Klein, 1979)

Table 11: Research Review of Decreased Baseflow in Urban Streams

Reference	Key Finding	Location
Finkenbine *et al.*, 2000	Summer base flow was uniformly low in 11 streams when IC reached 40% or greater.	Vancouver
Klein, 1979	Baseflow decreased as IC increased in Piedmont streams.	MD
Saravanapavan, 2002	Percentage of baseflow decreased linearly as IC increased for 13 subwatersheds of Shawsheen River watershed.	MA
Simmons and Reynolds, 1982	Dry weather flow dropped 20 to 85% after development in several urban watersheds on Long Island.	NY
Spinello and Simmons, 1992	Baseflow in two Long Island streams went dry as a result of urbanization.	NY
Konrad and Booth, 2002	No discernable trend over many decades in the annual seven day low flow discharge for 11 Washington streams.	WA
Wang *et al.*, 2001	Stream baseflow was negatively correlated with watershed IC in 47 small streams, with an apparent breakpoint at 8 to 12% IC.	WI
Evett *et al.*, 1994	No clear relationship between dry weather flow and urban and rural streams in 21 larger watersheds.	NC



Notes:

Carmans and Connetquot: Rural/suburban, watersheds are unsewered

Bellmore and Massapequa: Moderately urbanized, not sewered until 1989

Valley Stream and Pine Brook: Urbanized and densely populated watersheds, sewer systems completed in 1960s

1953-1964: Period of sanitary sewer construction in eastern Nassau County

1962-1966: Drought years

Figure 15: Baseflow Response to Urbanization in Long Island Streams (Spinello and Simmons, 1992)

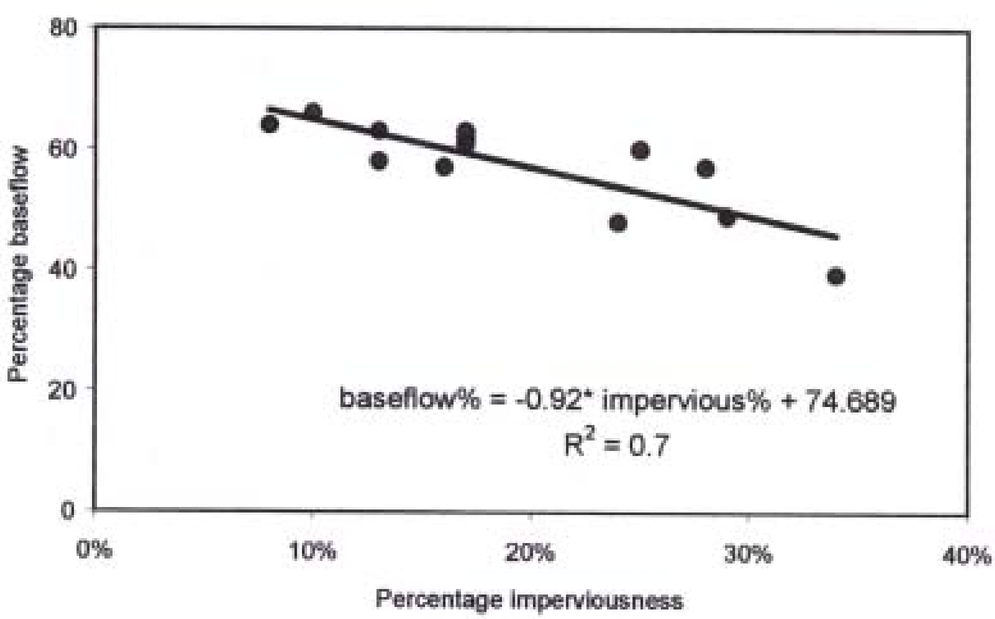


Figure 16: Relationship Between Percentage Baseflow and Percent IC in Massachusetts Streams (Saravanapan, 2002)

Finkebine *et al.* (2000) monitored summer baseflow in 11 streams near Vancouver, British Columbia and found that stream base flow was uniformly low due to decreased groundwater recharge in watersheds with more than 40% IC (Figure 17). Baseflow velocity also consistently decreased when IC increased (Figure 18). The study cautioned that other factors can affect stream baseflow, such as watershed geology and age of development.

Other studies, however, have not been able to establish a relationship between IC and declining baseflow. For example, a study in North Carolina could not conclusively determine that urbanization reduced baseflow in larger urban and suburban watersheds in that area (Evett *et al.*, 1994). In some cases, stream baseflow is supported by deeper aquifers or originate in areas outside the surface watershed boundary. In others, baseflow is augmented by leaking sewers, water pipes and irrigation return flows.

This appears to be particularly true in arid and semi-arid areas, where baseflow can actually increase in response to greater IC (Hollis, 1975). For instance, Crippen and Waananen (1969) found that Sharon Creek near San Francisco changed from an ephemeral stream into a perennial stream after urban development. Increased infiltration from lawn watering and return flow from sewage treatment plants are two common sources of augmented baseflows in these regions (Caraco, 2000a).

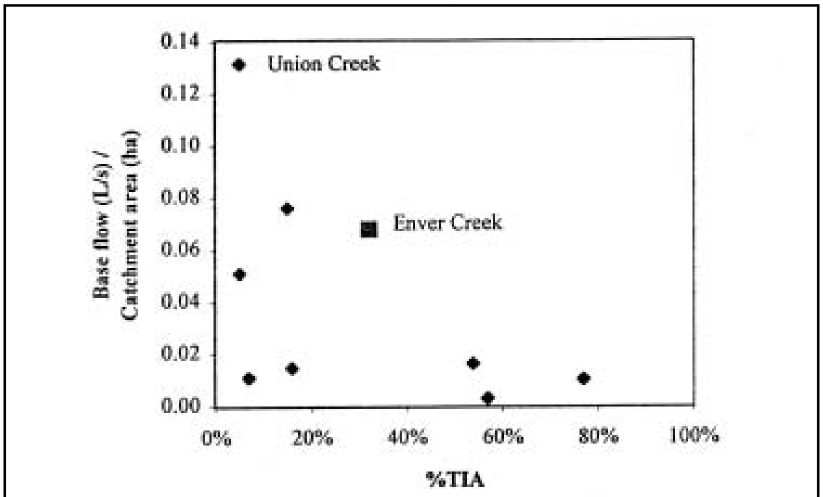


Figure 17: Effect of IC on Summer Baseflow in Vancouver Streams (Finkerbine *et al.*, 2000)

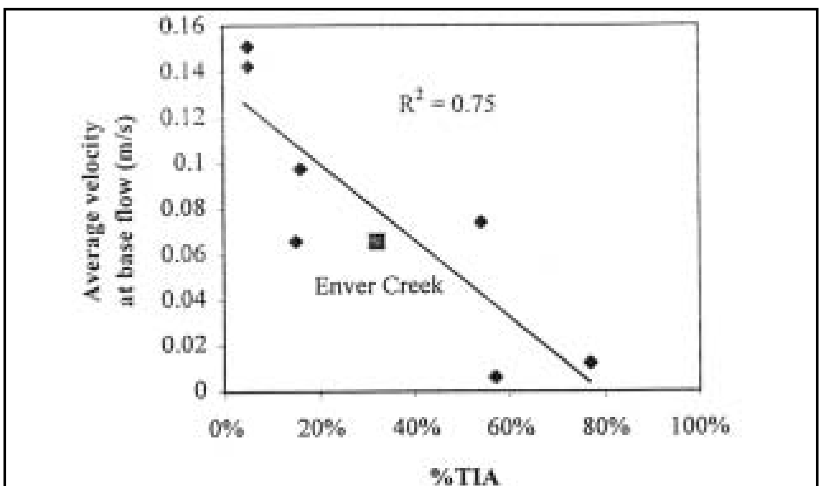


Figure 18: Effect of Watershed IC on Summer Stream Velocity in Vancouver Streams (Finkerbine *et al.*, 2000)

2.6 Conclusions

The changes in hydrology indicators caused by watershed urbanization include increased runoff volume; increased peak discharge; increased magnitude, frequency and duration of bankfull flows; flashier/less predictable flows; and decreased baseflow. Many studies support the direct relationship between IC and these indicators. However, at low levels of watershed IC, site-specific factors such as slope, soils, types of conveyance systems, age of development, and watershed dimensions often play a stronger role in determining a watershed's hydrologic response.

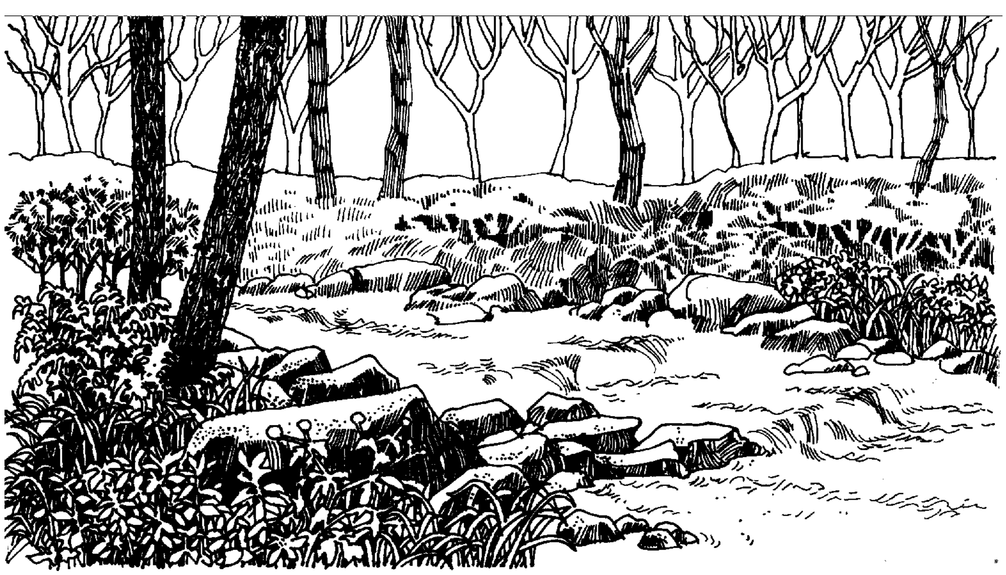
Overall, the following conclusions can be drawn from the relationship between watershed IC and hydrology indicators:

- Strong evidence exists for the direct relationship between watershed IC and increased stormwater runoff volume and peak discharge. These relationships are considered so strong that they have been incorporated into widely accepted engineering models.

- The relationship between IC and bankfull flow frequency has not been extensively documented, although abundant data exists for differences between urban and non-urban watersheds.

- The relationship between IC and declining stream flow is more ambiguous and appears to vary regionally in response to climate and geologic factors, as well as water and sewer infrastructure.

The changes in hydrology indicators caused by watershed urbanization directly influence physical and habitat characteristics of streams. The next chapter reviews how urban streams physically respond to the major changes to their hydrology.



Chapter 3: Physical Impacts of Impervious Cover

A growing body of scientific literature documents the physical changes that occur in streams undergoing watershed urbanization. This chapter discusses the impact of watershed development on various measures of physical habitat in urban stream channels and is organized as follows:

3.1 Difficulty in Measuring Habitat
3.2 Changes in Channel Geometry
3.3 Effect on Composite Indexes of Stream Habitat
3.4 Effect on Individual Elements of Stream Habitat
3.5 Increased Stream Warming
3.6 Alteration of Stream Channel Network
3.7 Conclusion

This chapter reviews the available evidence on stream habitat. We begin by looking at geomorphological research that has examined how the geometry of streams changes in response to altered urban hydrology. The typical response is an enlargement of the cross-sectional area of the stream channel through a process of channel incision, widening, or a combination of both. This process triggers an increase in bank and/or bed erosion that increases sediment transport from the stream, possibly for several decades or more.

Next, we examine the handful of studies that have evaluated the relationship between watershed development and composite indicators of stream habitat (such as the habitat Rapid Bioassessment Protocol, or RBP). In the fourth section, we examine the dozen studies that have evaluated how individual habitat elements respond to watershed development. These studies show a consistent picture. Generally, streams with low levels of IC have stable banks, contain considerable large woody debris (LWD) and possess complex habitat structure. As watershed IC increases, however, urban streambanks become increasingly unstable, streams lose LWD, and they develop a more simple and uniform habitat structure. This is typified by reduced pool depths, loss of pool and riffle sequences, reduced channel roughness and less channel sinuosity.

Water temperature is often regarded as a key habitat element, and the fifth section describes the stream warming effect observed in urban streams in six studies. The last section looks at the effect of watershed development on the stream channel network as a whole, in regard to headwater stream loss and the creation of fish barriers.

3.1 Difficulty in Measuring Habitat

The physical transformation of urban streams is perhaps the most conspicuous impact of watershed development. These dramatic physical changes are easily documented in sequences of stream photos with progressively greater watershed IC (see Figure 19). Indeed, the network of headwater stream channels generally disappears when watershed IC exceeds 60% (CWP).

3.1.1 The Habitat Problem

It is interesting to note that while the physical impacts of urbanization on streams are widely accepted, they have rarely been documented by the research community. As a consequence, no predictive models exist to quantify how physical indicators of stream habitat will decline in response to watershed IC, despite the fact that most would agree that some kind of decline is expected (see Table 12).

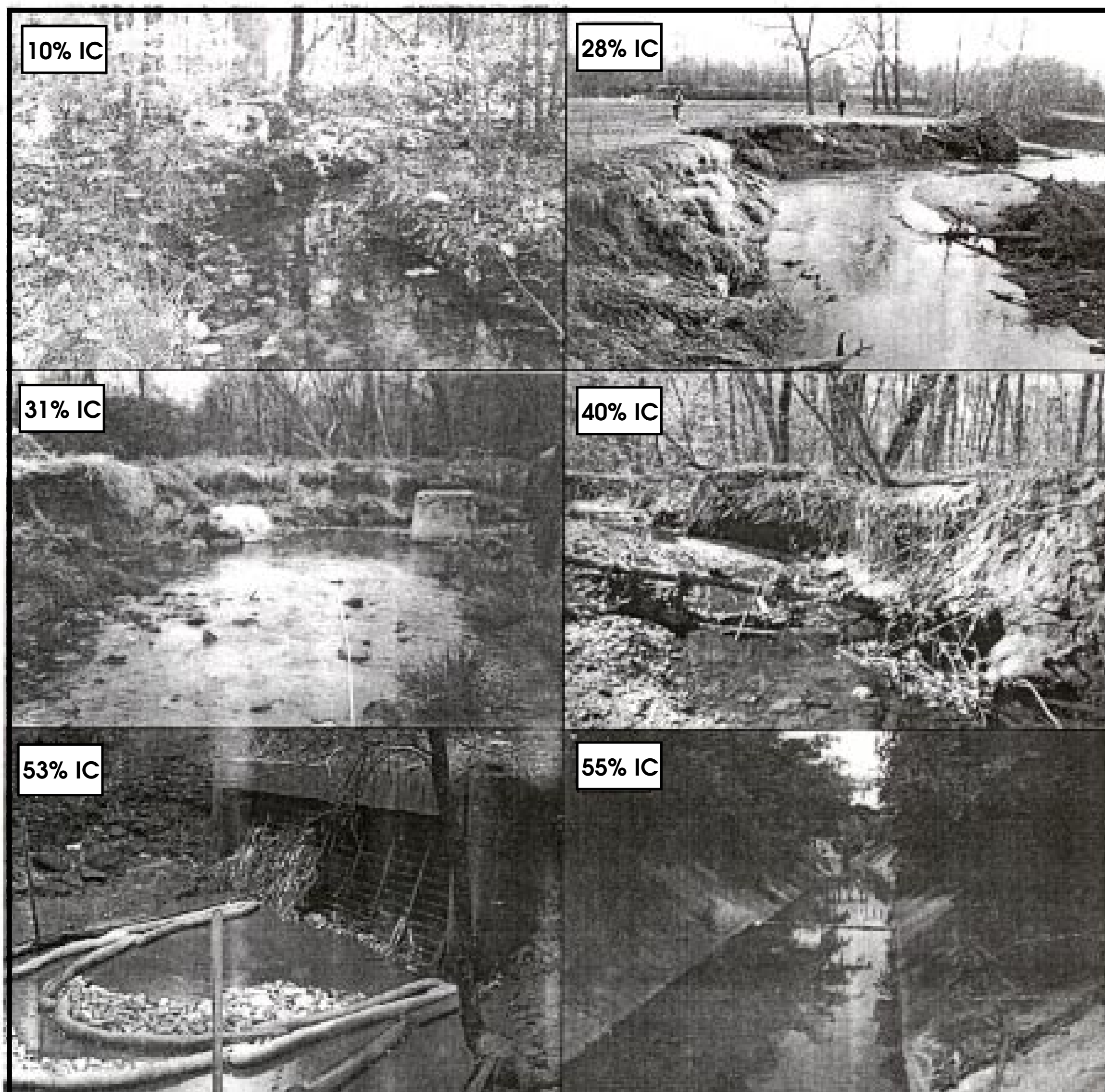


Figure 19: Urban Stream Channels with Progressively Greater IC

The main reason for this gap is that "habitat" is extremely hard to define, and even more difficult to measure in the field. Most indices of physical habitat involve a visual and qualitative assessment of 10 or more individual habitat elements that are perceived by fishery and stream biologists to contribute to quality stream habitat. Since these indices include many different habitat elements, each of which is given equal weight, they have not been very useful in discriminating watershed effects (Wang *et al.*, 2001).

Table 12: Physical Impacts of Urbanization on Streams
Specific Impacts
Sediment transport modified Channel enlargement Channel incision Stream embeddedness Loss of large woody debris Changes in pool/riffle structure Loss of riparian cover Reduced channel sinuosity Warmer in-stream temperatures Loss of cold water species and diversity Channel hardening Fish blockages Loss of 1st and 2nd order streams through storm drain enclosure

Researchers have had greater success in relating individual habitat elements to watershed conditions, such as large woody debris (LWD), embeddedness, or bank stability. Even so, direct testing has been limited, partly because individual habitat elements are hard to measure and are notoriously variable in both space and time. Consider bank stability for a moment. It would be quite surprising to see a highly urban stream that did not have unstable banks. Yet, the hard question is exactly how would bank instability be quantitatively measured? Where would it be measured — at a point, a cross-section, along a reach, on the left bank or the right?

Geomorphologists stress that no two stream reaches are exactly alike, due to differences in gradient, bed material, sediment transport, hydrology, watershed history and many other factors. Consequently, it is difficult to make controlled comparisons among different streams. Indeed, geomorphic theory stresses that individual stream reaches respond in a highly dynamic way to changes in watershed hydrology and sediment transport, and can take several decades to fully adjust to a new equilibrium.

Returning to our example of defining bank stability, how might our measure of bank instability change over time as its watershed gradually urbanizes, is built out, and possibly reaches a new equilibrium over several decades? It is not very surprising that the effect of watershed development on stream habitat is widely observed, yet rarely measured.

3.2 Changes in Stream Geometry

As noted in the last chapter, urbanization causes an increase in the frequency and duration of bankfull and sub-bankfull flow events in streams. These flow events perform more “effective work” on the stream channel, as defined by Leopold (1994). The net effect is that an urban stream channel is exposed to more shear stress above the critical threshold needed to move bank and bed sediments (Figure 20). This usually triggers a cycle of active bank erosion and greater sediment transport in urban streams. As a consequence, the stream channel adjusts by expanding its cross-sectional area, in order to effectively accommodate greater flows and sediment supply. The stream channel can expand by incision, widening, or both. Incision refers to stream down-cutting through the streambed, whereas widening refers to lateral erosion of the stream bank and its flood plain (Allen and Narramore, 1985; Booth, 1990; Morisawa and LaFlure, 1979).

3.2.1 Channel Enlargement

A handful of research studies have specifically examined the relationship between watershed development and stream channel enlargement (Table 13). These studies indicate that stream cross-sectional areas can enlarge by as much as two to eight times in response to urbanization, although the process is complex and may take several decades to complete (Pizzuto *et al.*, 2000; Caraco, 2000b; Hammer, 1972). An example of channel enlargement is provided in Figure 21, which shows how a stream cross-section in Watts Branch near Rockville, Maryland has expanded in response to nearly five decades of urbanization (i.e., watershed IC increased from two to 27%).

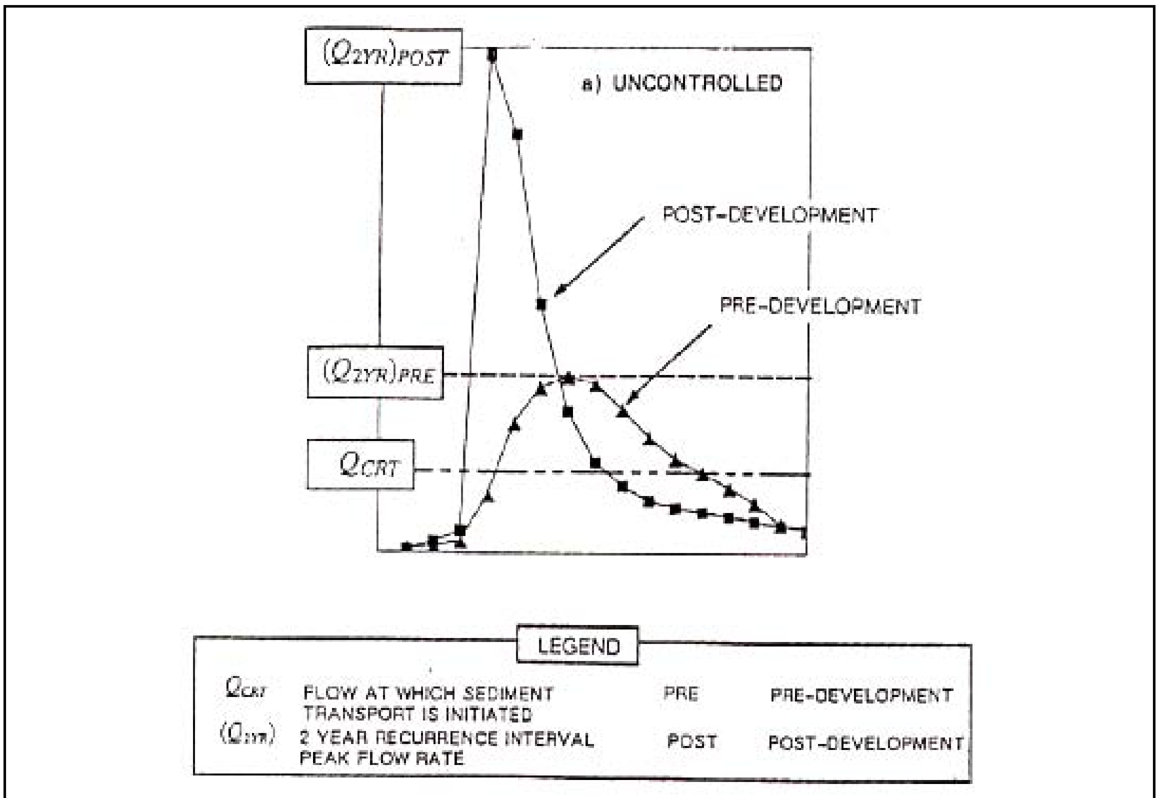


Figure 20: Increased Shear Stress from a Hydrograph (MacRae and Rowney, 1992)

Table 13: Research Review of Channel Enlargement and Sediment Transport in Urban Streams

Reference	Key Finding	Location
% IC used as Indicator		
Caraco, 2000b	Reported enlargement in ratios of 1.5 to 2.2 for 10 stream reaches in Watts Branch and computed ultimate enlargement ratios of 2.0	MD
MacCrae and De Andrea, 1999	Introduced the concept of ultimate channel enlargement based on watershed IC and channel characteristics.	Ontario, TX
Morse, 2001	Demonstrated increased erosion rates with increases in IC (channels were generally of the same geomorphic type).	ME
Urbanization Used as Indicator		
Allen and Narramore, 1985	Enlargement ratios in two urban streams ranged from 1.7 to 2.4.	TX
Bledsoe, 2001	Reported that channel response to urbanization depends on other factors in addition to watershed IC including geology, vegetation, sediment and flow regimes.	N/A
Booth and Henshaw, 2001	Evaluated channel cross section erosion rates and determined that these rates vary based on additional factors including the underlying geology, age of development and gradient.	WA
Hammer, 1972	Enlargement ratios ranged from 0.7 to 3.8 in urban watersheds.	PA
Neller, 1989	Enlargement ratios in small urban catchments ranged from two to 7.19, the higher enlargement ratios were primarily from incision occurring in small channels.	Australia
Pizzuto *et al.*, 2000	Evaluated channel characteristics of paired urban and rural streams and demonstrated median bankfull cross sectional increase of 180%. Median values for channel sinuosity were 8% lower in urban streams; Mannings N values were found to be 10% lower in urban streams.	PA
Hession *et al.*, *in press*	Bankfull widths for urban streams were significantly wider than non-urban streams in 26 paired streams. Forested reaches were consistently wider than non-forested reaches in urban streams.	MD, DE, PA
Dartiguenave *et al.*, 1997	Bank erosion accounted for up to 75% of the sediment transport in urban watersheds.	TX
Trimble, 1997	Demonstrated channel enlargement over time in an urbanizing San Diego Creek; Bank erosion accounted for over 66% of the sediment transport.	CA

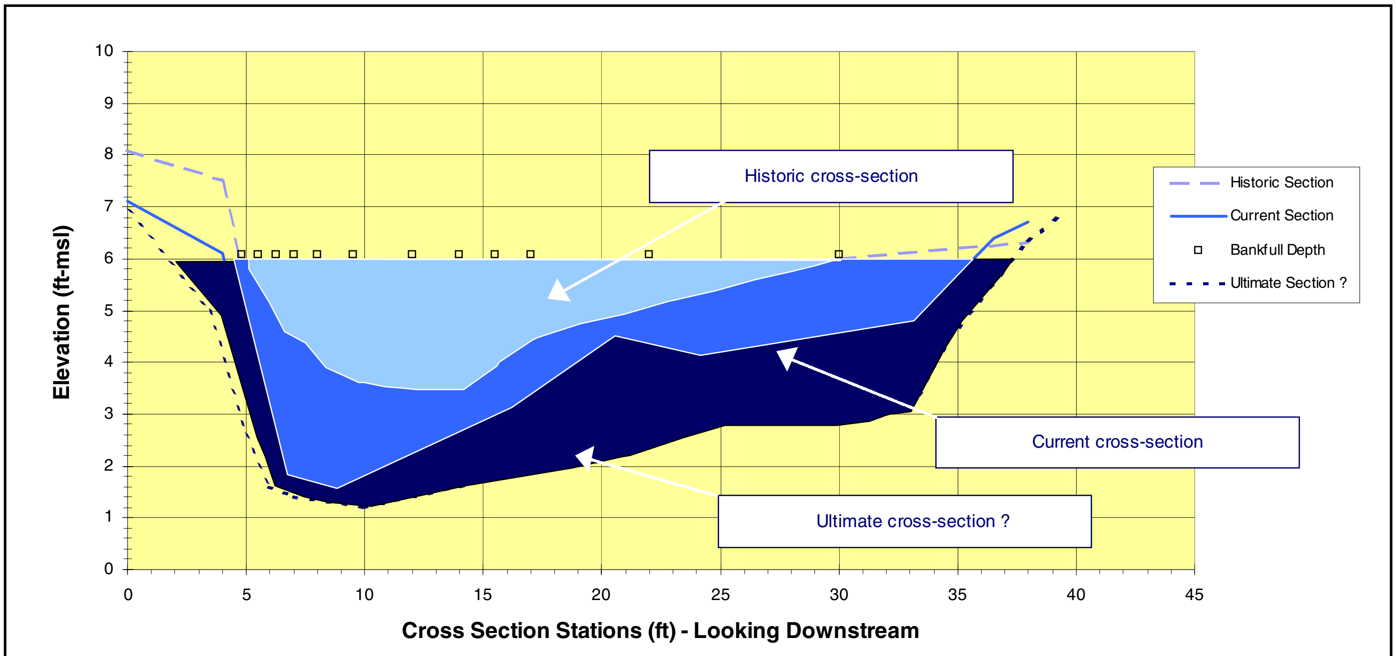


Figure 21: Stream Channel Enlargement in Watts Branch, MD 1950-2000 (Caraco, 2000b)

Some geomorphologists suggest that urban stream channels will reach an "ultimate enlargement" relative to pre-developed channels (MacRae and DeAndrea, 1999) and that this can be predicted based on watershed IC, age of development, and the resistance of the channel bed and banks. A relationship between ultimate stream channel enlargement and watershed IC has been developed for alluvial streams in Texas, Vermont and Maryland (Figure 22). Other geomorphologists such as Bledsoe (2001) and Booth and Henshaw (2001) contend that channel response to urbanization is more complex, and also depends on geology, grade control, stream gradient and other factors.

Channel incision is often limited by grade control caused by bedrock, cobbles, armored substrates, bridges, culverts and pipelines. These features can impede the downward erosion of the stream channel and thereby limit the incision process. Stream incision can become severe in streams that have softer substrates such as sand, gravel and clay (Booth, 1990). For example, Allen and Narramore (1985) showed that channel enlargement in chalk channels was 12 to 67% greater than in shale channels near Dallas, Texas. They attributed the differences to the softer substrate, greater velocities and higher shear stress in the chalk channels.

Neller (1989) and Booth and Henshaw (2001) also report that incised urban stream channels possess cross-sectional areas that are larger than would be predicted based on watershed area or discharge alone. This is due to the fact that larger floods are often contained within the stream channel rather than the floodplain. Thus, incised channels often result in greater erosion and geomorphic change. In general, stream conditions that can foster incision include erodible substrates, moderate to high stream gradients, and an absence of grade control features.

Channel widening occurs more frequently when streams have grade control and the stream has cut into its bank, thereby expanding its cross-sectional area. Urban stream channels often have artificial grade controls caused by frequent culverts and road crossings. These grade controls often cause localized sediment deposition that can reduce the capacity of culverts and bridge crossings to pass flood waters.

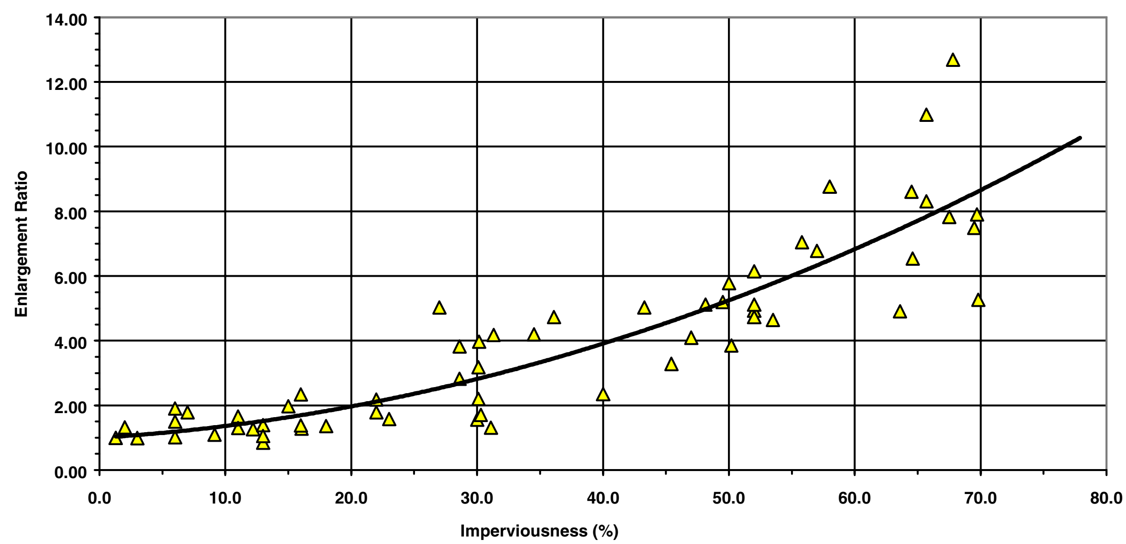


Figure 22: Ultimate Channel Enlargement in MD, UT and TX Alluvial Streams (MacRae and DeAndrea, 1999 and CWP, 2001b)

The loss of flood plain and riparian vegetation has been strongly associated with watershed urbanization (May *et al.*, 1997). A few studies have shown that the loss of riparian trees can result in increased erosion and channel migration rates (Beeson and Doyle, 1995 and Allmendinger *et al.*, 1999). For example, Beeson and Doyle (1995) found that meander bends with vegetation were five times less likely to experience significant erosion from a major flood than non-vegetated meander bends.Hession *et al.* (in press) observed that forested reaches consistently had greater bankfull widths than non-forested reaches in a series of urban streams in Pennsylvania, Maryland and Delaware.

3.2.2 Effect of Channel Enlargement on Sediment Yield

Regardless of whether a stream incises, widens, or does both, it will greatly increase sediment transport from the watershed due to erosion. Urban stream research conducted in California and Texas suggests that 60 to 75% of the sediment yield of urban watersheds can be derived from channel erosion (Trimble, 1997 and Dartingunave *et al.*, 1997) This can be compared to estimates for rural streams where channel erosion accounts for only five to 20% of the annual sediment yield (Collins *et al.*, 1997 and Walling and Woodward, 1995).

Some geomorphologists speculate that urban stream channels will ultimately adjust to their post-development flow regime and sediment supply. Finkenbine *et al.* (2000) observed these conditions in Vancouver streams, where study streams eventually stabilized two decades after the watersheds were fully developed. In older urban streams, reduced sediment transport can be expected when urbanization has been completed. At this point, headwater stream channels are replaced by storm drains and pipes, which can transport less sediment. The lack of available sediment may cause downstream channel erosion, due to the diminished sediment supply found in the stream.

3.3 Effect on Composite Measures of Stream Habitat

Composite measures of stream habitat refer to assessments such as EPA's Habitat Rapid Bioassessment Protocol (RBP) that combine multiple habitat elements into a single score or index (Barbour *et al*., 1999). For example, the RBP requires visual assessment of 10 stream habitat elements, including embeddedness, epifaunal substrate quality, velocity/depth regime, sediment deposition, channel flow status, riffle frequency, bank stabilization, streambank vegetation and riparian vegetation width. Each habitat element is qualitatively scored on a 20 point scale, and each element is weighted equally to derive a composite score for the stream reach.

To date, several studies have found a relationship between declining composite habitat indicator scores and increasing watershed IC in different eco-regions of the United States. A typical pattern in the composite habitat scores is provided for headwater streams in Maine (Morse, 2001; Figure 23). This general finding has been reported in the mid-Atlantic, Northeast and the Northwest (Black and Veatch, 1994; Booth and Jackson, 1997; Hicks and Larson, 1997; Maxted and Shaver, 1997; Morse, 2001; Stranko and Rodney, 2001).

However, other researchers have found a much weaker relationship between composite habitat scores and watershed IC. Wang and his colleagues (2001) found that composite habitat scores were not correlated with watershed IC in Wisconsin streams, although it was correlated with individual habitat elements, such as streambank erosion. They noted that many agricultural and rural streams had fair to poor composite habitat scores, due to poor riparian management and sediment deposition. The same basic conclusion was also reported for streams of the Maryland Piedmont (MNCPPC, 2000).

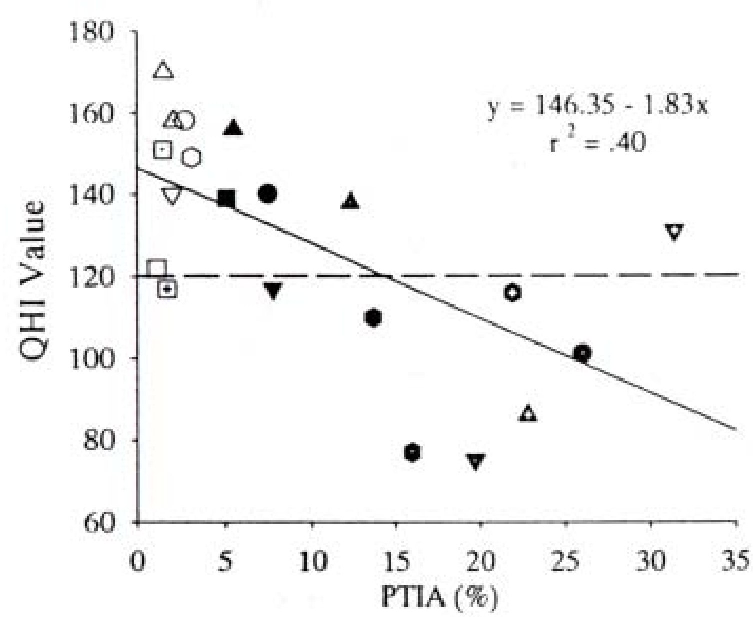


Figure 23: Relationship Between Habitat Quality and IC in Maine Streams (Morse, 2001)

3.4 Effect on Individual Elements of Stream Habitat

Roughly a dozen studies have examined the effect of watershed development on the degradation of individual stream habitat features such as bank stability, embeddedness, riffle/pool quality, and loss of LWD (Table 14). Much of this data has been acquired from the Pacific Northwest, where the importance of such habitat for migrating salmon has been a persistent management concern.

3.4.1 Bank Erosion and Bank Stability

It is somewhat surprising that we could only find one study that related bank stability or bank erosion to watershed IC. Conducted by Booth (1991) in the streams of the Puget Sound lowlands, the study reported that stream banks were consistently rated as stable in watersheds with less than 10% IC, but became progressively more unstable above this threshold. Dozens of stream assessments have found high rates of bank erosion in urban streams, but none, to our knowledge, has systematically related the prevalence or severity of bank erosion to watershed IC. As noted earlier, this may reflect the lack of a universally recognized method to measure comparative bank erosion in the field.

3.4.2 Embeddedness

Embeddedness is a term that describes the extent to which the rock surfaces found on the stream bottom are filled in with sand, silts and clay. In a healthy stream, the interstitial pores between cobbles, rock and gravel generally lack fine sediments, and are an active habitat zone and detrital processing area. The increased sediment transport in urban streams can rapidly fill up these pores in a process known as embedding. Normally, embeddedness is visually measured in riffle zones of streams. Riffles tend to be an important habitat for aquatic insects and fish (such as darters and sculpins). Clean stream substrates are also critical to trout and salmon egg incubation and embryo development. May *et al.* (1997) demonstrated that the percent of fine sediment particles in riffles generally increased with watershed IC (Figure 24). However, Finkenbine *et al.* (2000) reported that embeddedness eventually decreased slightly after watershed land use and sediment transport had stabilized for 20 years.

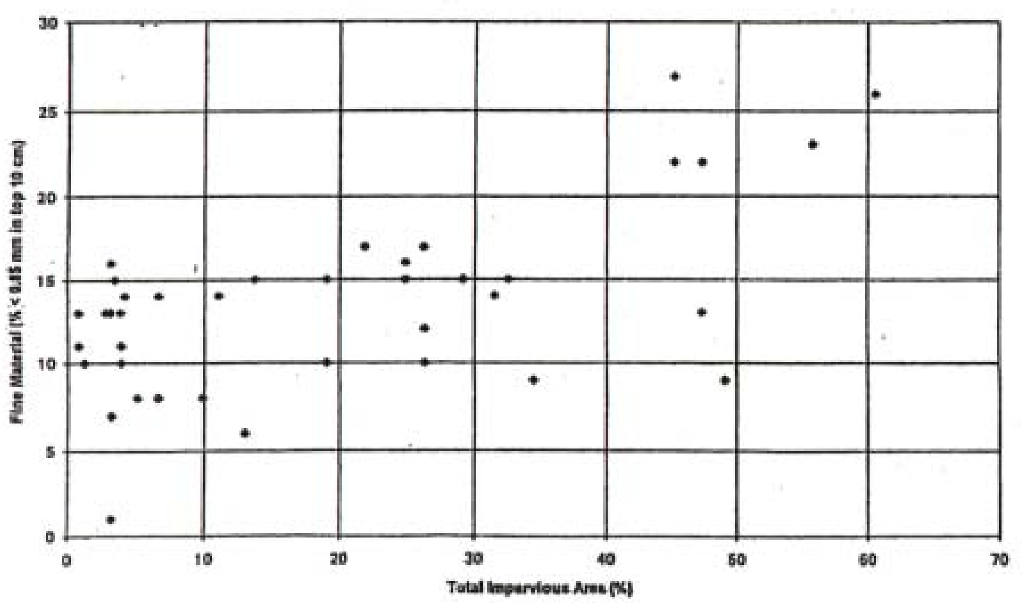


Figure 24: Fine Material Sediment Deposition as a Function of IC in Pacific Northwest Streams (Horner *et al.*, 1997)

Table 14: Research Review of Changes in Urban Stream Habitat		
Reference	**Key Finding**	**Location**
% IC Used as Indicator		
Black & Veatch, 1994	Habitat scores were ranked as poor in five subwatersheds that had greater than 30% IC.	MD
Booth and Jackson, 1997	Increase in degraded habitat conditions with increases in watershed IC.	WA
Hicks and Larson, 1997	Reported a reduction in composite stream habitat indices with increasing watershed IC.	MA
May *et al.*, 1997	Composite stream habitat declined most rapidly during the initial phase of the watershed urbanization, when percent IC exceeded the 5-10% range.	WA
Stranko and Rodney, 2001	Composite index of stream habitat declined with increasing watershed IC in coastal plain streams.	MD
Wang *et al.*, 2001	Composite stream habitat scores were not correlated with watershed IC in 47 small watersheds, although channel erosion was. Non-urban watersheds were highly agricultural and often lacked riparian forest buffers.	WI
MNCPPC, 2000	Reported that stream habitat scores were not correlated with IC in suburban watersheds.	MD
Morse, 2001	Composite habitat values tended to decline with increases in watershed IC.	ME
Booth, 1991	Channel stability and fish habitat quality declined rapidly after 10% watershed IC.	WA
Booth *et al.*, 1997	Decreased LWD with increased IC.	PNW
Finkenbine *et al.*, 2000	LWD was scarce in streams with greater than 20% IC in Vancouver.	B.C.
Horner & May, 1999	When IC levels were >5%, average LWD densities fell below 300 pieces/kilometer.	PNW
Horner *et al.*, 1997	Interstitial spaces in streambed sediments begin to fill with increasing watershed IC.	PNW
Urbanization Used as Indicator		
Dunne and Leopold, 1978	Natural channels replaced by storm drains and pipes; increased erosion rates observed downstream.	MD
May *et al.*, 1997	Forested riparian corridor width declines with increased watershed IC.	PNW
MWCOG, 1992	Fish blockages caused by bridges and culverts noted in urban watersheds.	D.C.
Pizzuto *et al.*, 2000	Urban streams had reduced pool depth, roughness, and sinuosity, compared to rural streams; Pools were 31% shallower in urban streams compared to non-urban ones.	PA
Richey, 1982	Altered pool/riffle sequence observed in urban streams.	WA
Scott *et al.*, 1986	Loss of habitat diversity noted in urban watersheds.	PNW
Spence *et al.*, 1996	Large woody debris is important for habitat diversity and anadromous fish.	PNW

3.4.3 Large Woody Debris (LWD)

LWD is a habitat element that describes the approximate volume of large woody material (< four inches in diameter) found in contact with the stream. The presence and stability of LWD is an important habitat parameter in streams. LWD can form dams and pools, trap sediment and detritus, stabilize stream channels, dissipate flow energy, and promote habitat complexity (Booth *et al.,* 1997). LWD creates a variety of pool features (plunge, lateral, scour and backwater); short riffles; undercut banks; side channels; and a range of water depths (Spence *et al.*, 1996). Urban streams tend to have a low supply of LWD, as increased stormwater flows transport LWD and clears riparian areas. Horner *et al.* (1997) presents evidence from Pacific Northwest streams that LWD decreases in response to increasing watershed IC (Figure 25).

3.4.4 Changes in Other Individual Stream Parameters

One of the notable changes in urban stream habitat is a decrease in pool depth and a general simplification of habitat features such as pools, riffles and runs. For example, Richey (1982) and Scott *et al.* (1986) reported an increase in the prevalence of glides and a corresponding altered riffle/pool sequence due to urbanization. Pizzuto *et al.* (2000) reported a median 31% decrease in pool depth in urban streams when compared to forested streams. Pizzuto *et al.* also reported a modest decrease in channel sinuosity and channel roughness in the same urban streams in Pennsylvania.

Several individual stream habitat parameters appear to have received no attention in urban stream research to date. These parameters include riparian shading, wetted perimeter, various measures of velocity/depth regimes, riffle frequency, and sediment deposition in pools. More systematic monitoring of these individual stream habitat parameters may be warranted.

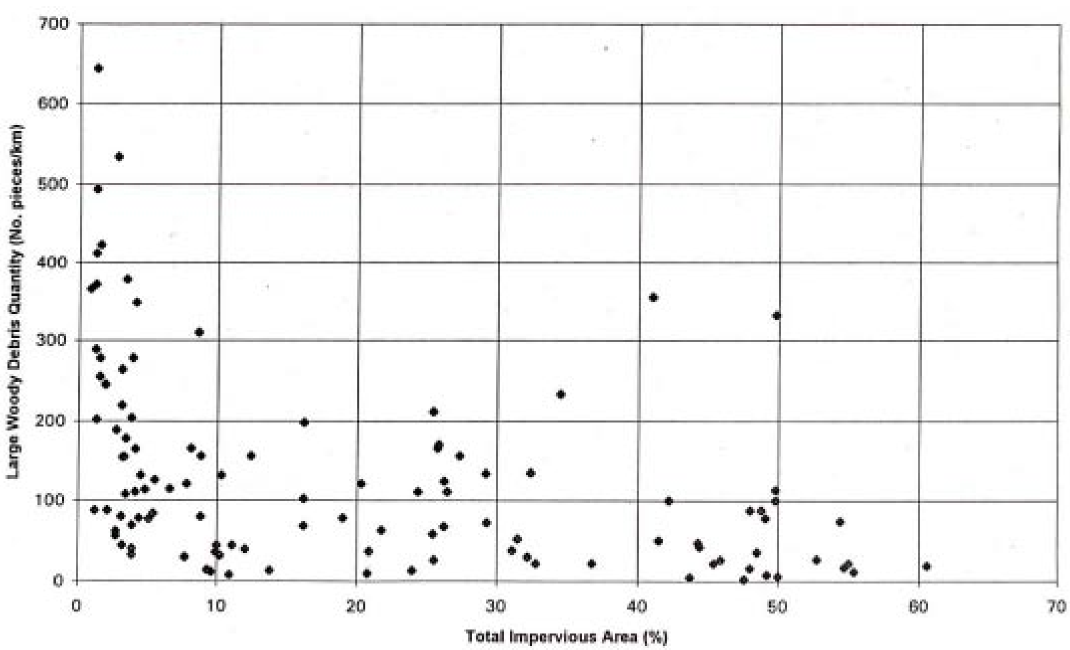


Figure 25: LWD as a Function of IC in Puget Sound Streams (Horner *et al.*, 1997)

3.5 Increased Stream Warming

IC directly influences our local weather in urban areas. This effect is obvious to anyone walking across a parking lot on a hot summer day, when temperatures often reach a scorching 110 to 120 degrees F. Parking lots and other hard surfaces tend to absorb solar energy and release it slowly. Furthermore, they lack the normal cooling properties of trees and vegetation, which act as natural air conditioners. Finally, urban areas release excess heat as a result of the combustion of fossil fuels for heating, cooling and transportation. As a result, highly urban areas tend to be much warmer than their rural counterparts and are known as urban heat islands. Researchers have found that summer temperatures tend to be six to eight degrees F warmer in the summer and two to four degrees F warmer during the winter months.

Water temperature in headwater streams is strongly influenced by local air temperatures. Summer temperatures in urban streams have been shown to increase by as much as five to 12 degrees F in response to watershed development (Table 15). Increased water temperatures can preclude temperature-sensitive species from being able to survive in urban streams.

Figure 26 shows the stream warming phenomenon in small headwater streams in the Maryland Piedmont.

Galli (1990) reported that stream temperatures throughout the summer increased in urban watersheds. He monitored five headwater streams in the Maryland Piedmont with different levels of IC. Each urban stream had mean temperatures that were consistently warmer than a forested reference stream, and stream warming appeared to be a direct function of watershed IC. Other factors, such as lack of riparian cover and the presence of ponds, were also demonstrated to amplify stream warming, but the primary contributing factor appeared to be watershed IC.

Johnson (1995) studied how stormwater influenced an urban trout stream in Minnesota and reported up to a 10 degree F increase in stream water temperatures after summer storm events. Paul *et al.* (2001) evaluated stream temperatures for 30 subwatersheds to the Etowah River in Georgia, which ranged from five to 61% urban land. They found a correlation between summer daily mean water temperatures and the percentage of urban land in a subwatershed.

Table 15: Research Review of Thermal Impacts in Urban Streams

Reference	Key Finding	Location
%IC Used as Indicator		
Galli, 1990	Increase in stream temperatures of five to 12 degrees Fahrenheit in urban watersheds; stream warming linked to IC.	MD
Urbanization Used as Indicator		
Johnson, 1995	Up to 10 degrees Fahrenheit increases in stream temperatures after summer storm events in an urban area	MN
LeBlanc *et al.*, 1997	Calibrated a model predicting stream temperature increase as a result of urbanization	Ontario
MCDEP, 2000	Monitoring effect of urbanization and stormwater ponds on stream temperatures revealed stream warming associated with urbanization and stormwater ponds	MD
Paul *et al.*, 2001	Daily mean stream temperatures in summer increased with urban land use	GA

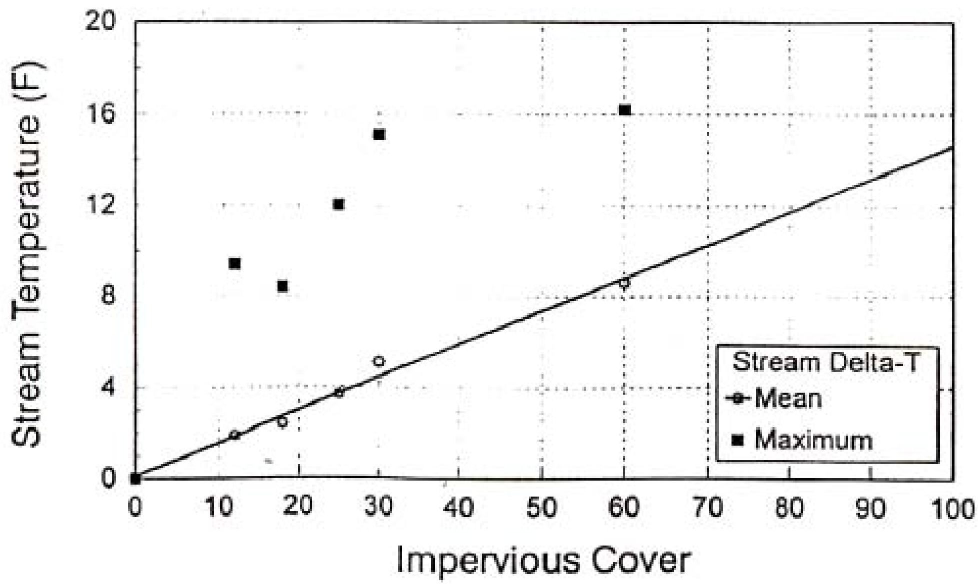


Figure 26: Stream Temperature Increase in Response to IC in Maryland Piedmont Streams (Galli, 1990)

Discharges from stormwater ponds can also contribute to stream warming in urban watersheds. Three studies highlight the temperature increase that can result from stormwater ponds. A study in Ontario found that baseflow temperatures below wet stormwater ponds increased by nine to 18 degrees F in the summer (SWAMP, 2000a, b). Oberts (1997) also measured change in the baseflow temperature as it flowed through a wetland/wet pond system in Minnesota. He concluded that the temperature had increased by an average of nine degrees F during the summer months. Galli (1988) also observed a mean increase of two to 10 degrees F in four stormwater ponds located in Maryland.

3.6 Alteration of Stream Channel Networks

Urban stream channels are often severely altered by man. Channels are lined with rip rap or concrete, natural channels are straightened, and first order and ephemeral streams are enclosed in storm drain pipes. From an engineering standpoint, these modifications rapidly convey flood waters downstream and locally stabilize stream banks. Cumulatively, however, these modifications can have a dramatic effect on the length and habitat quality of headwater stream networks.

3.6.1 Channel Modification

Over time, watershed development can alter or eliminate a significant percentage of the perennial stream network. In general, the loss of stream network becomes quite extensive when watershed IC exceeds 50%. This loss is striking when pre- and post-development stream networks are compared (Figure 27). The first panel illustrates the loss of stream network over time in a highly urban Northern Virginia watershed; the second panel shows how the drainage network of Rock Creek has changed in response to watershed development.

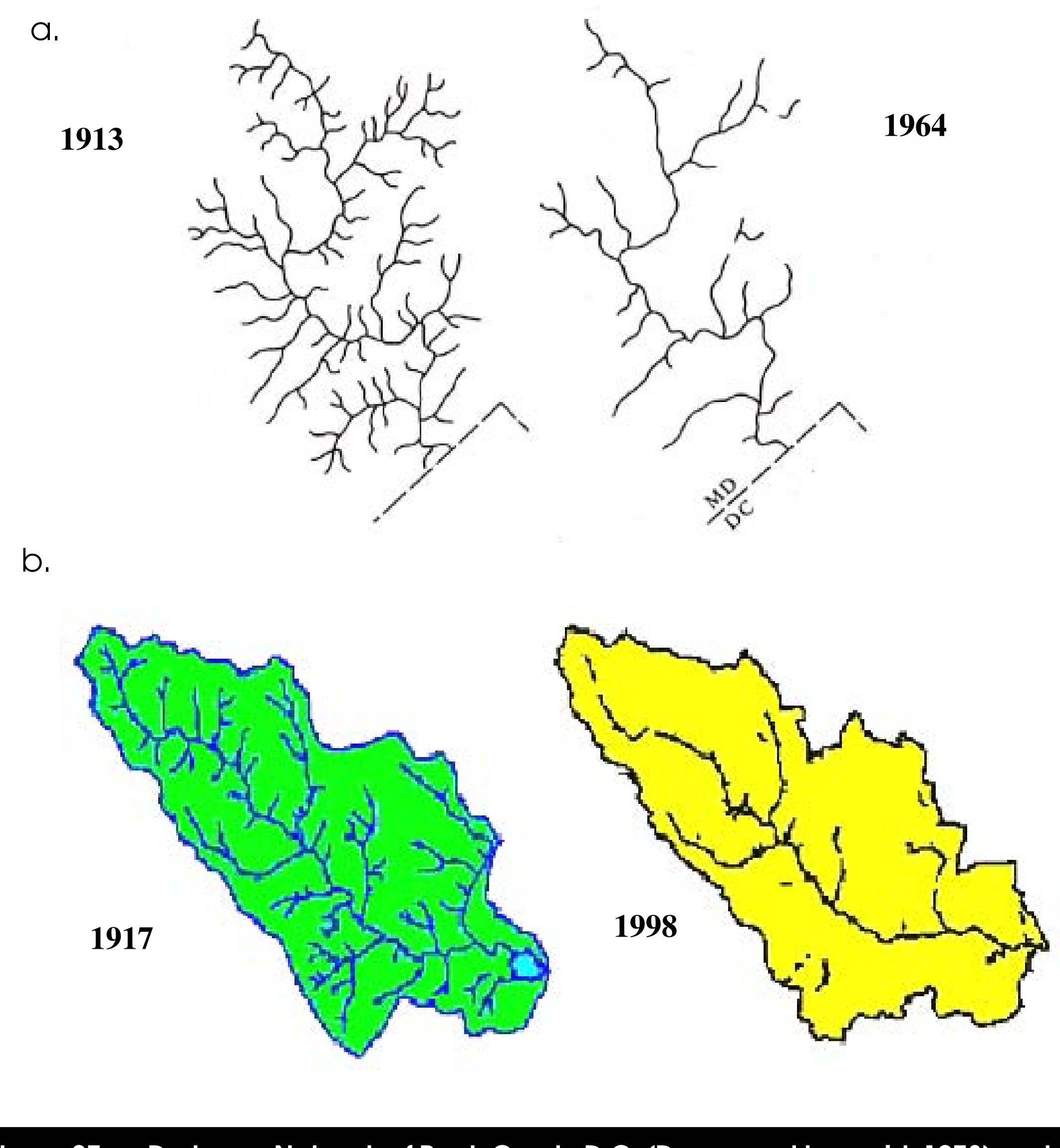


Figure 27: a. Drainage Network of Rock Creek, D.C. (Dunne and Leopold, 1978) and b. Drainage Network of Four Mile Run, VA Before and After Urbanization (NVRC, 2001)

In a national study of 269 gaged urban watersheds, Sauer *et al.* (1983) observed that channelization and channel hardening were important watershed variables that control peak discharge rates. The channel modifications increase the efficiency with which runoff is transported through the stream channel, increasing critical shear stress velocities and causing downstream channel erosion.

3.6.2 Barriers to Fish Migration

Infrastructure such as bridges, dams, pipelines and culverts can create partial or total barriers to fish migration and impair the ability of fish to move freely in a watershed. Blockages can have localized effects on small streams where non-migratory fish species can be prevented from re-colonizing upstream areas after acutely toxic events. The upstream movement of anadromous fish species such as shad, herring, salmon and steelhead can also be blocked by these barriers. Figure 28 depicts the prevalence of fish barriers in the Anacostia Watershed (MWCOG, 1992).

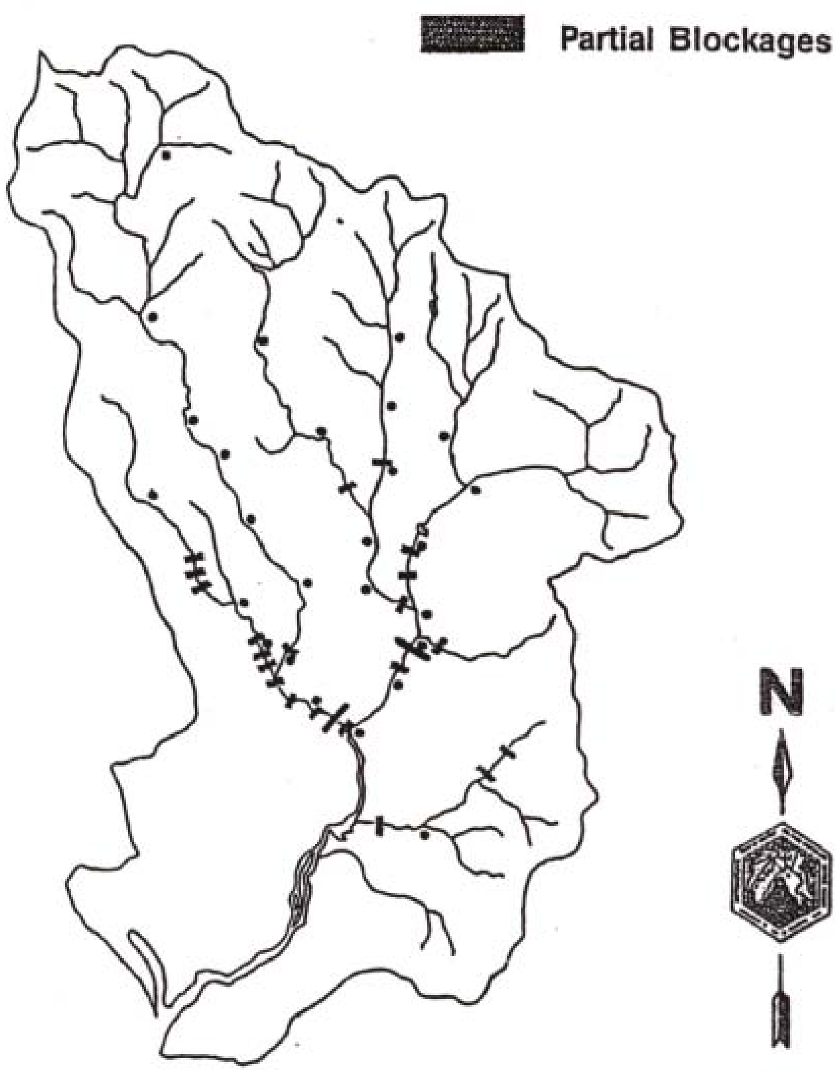


Figure 28: Fish Migration Barriers in the Anacostia Watershed of D.C. and MD (MWCOG, 1992)

3.7 Conclusion

Watershed development and the associated increase in IC have been found to significantly degrade the physical habitat of urban streams. In alluvial streams, the effects of channel enlargement and sediment transport can be severe at relatively low levels of IC (10 to 20%). However, the exact response of any stream is also contingent upon a combination of other physical factors such as geology, vegetation, gradient, the age of development, sediment supply, the use and design of stormwater treatment practices, and the extent of riparian buffers (Bledsoe, 2001).

Despite the uncertainty introduced by these factors, the limited geomorphic research to date suggests that physical habitat quality is almost always degraded by higher levels of watershed IC. Even in bedrock-controlled channels, where sediment transport and channel enlargement may not be as dramatic, researchers have noted changes in stream habitat features, such as embeddedness, loss of LWD, and stream warming.

Overall, the following conclusions can be made about the influence of watershed development on the physical habitat of urban streams:

- The major changes in physical habitat in urban streams are caused by the increased frequency and duration of bankfull and sub-bankfull discharges, and the attendant changes in sediment supply and transport. As a consequence, many urban streams experience significant channel enlargement. Generally, channel enlargement is most evident in alluvial streams.

- Typical habitat changes observed in urban streams include increased embeddedness, reduced supply of LWD, and simplification of stream habitat features such as pools, riffles and runs, as well as reduced channel sinuosity.

- Stream warming is often directly linked to watershed development, although more systematic subwatershed sampling is needed to precisely predict the extent of warming.

- Channel straightening, hardening and enclosure and the creation of fish barriers are all associated with watershed development. More systematic research is needed to establish whether these variables can be predicted based on watershed IC.

- In general, stream habitat diminishes at about 10% watershed IC, and becomes severely degraded beyond 25% watershed IC.

While our understanding of the relationship between stream habitat features and watershed development has improved in recent years, the topic deserves greater research in three areas. First, more systematic monitoring of composite habitat variables needs to be conducted across the full range of watershed IC. In particular, research is needed to define the approximate degree of watershed IC where urban streams are transformed into urban drainage systems.

Second, additional research is needed to explore the relationship between watershed IC and individual and measurable stream habitat parameters, such as bank erosion, channel sinuosity, pool depth and wetted perimeter. Lastly, more research is needed to determine if watershed treatment such as stormwater practices and stream buffers can mitigate the impacts of watershed IC on stream habitat. Together, these three research efforts could provide a technical foundation to develop a more predictive model of how watershed development influences stream habitat.

Chapter 4: Water Quality Impacts of Impervious Cover

This chapter presents information on pollutant concentrations found in urban stormwater runoff based on a national and regional data assessment for nine categories of pollutants. Included is a description of the Simple Method, which can be used to estimate pollutant loads based on the amount of IC found in a catchment or subwatershed. This chapter also addresses specific water quality impacts of stormwater pollutants and explores research on the sources and source areas of stormwater pollutants.

This chapter is organized as follows:

4.1 Introduction
4.2 Summary of National and Regional Stormwater Pollutant Concentration Data
4.3 Relationship Between Pollutant Loads and IC: The Simple Method
4.4 Sediment
4.5 Nutrients
4.6 Trace Metals
4.7 Hydrocarbons (PAH and Oil and Grease)
4.8 Bacteria and Pathogens
4.9 Organic Carbon
4.10 MTBE
4.11 Pesticides
4.12 Deicers
4.13 Conclusion

4.1 Introduction

Streams are usually the first aquatic system to receive stormwater runoff, and their water quality can be compromised by the pollutants it contains. Stormwater runoff typically contains dozens of pollutants that are detectable at some concentration, however small. Simply put, any pollutant deposited or derived from an activity on land will likely end up in stormwater runoff, although certain pollutants are consistently more likely to cause water quality problems in receiving waters. Pollutants that are frequently found in stormwater runoff can be grouped into nine broad categories: sediment, nutrients, metals, hydrocarbons, bacteria and pathogens, organic carbon, MTBE, pesticides, and deicers.

The impact that stormwater pollutants exert on water quality depends on many factors, including concentration, annual pollutant load, and category of pollutant. Based on nationally reported concentration data, there is considerable variation in stormwater pollutant concentrations. This variation has been at least partially attributed to regional differences, including rainfall and snowmelt. The volume and regularity of rainfall, the length of snow accumulation, and the rate of snowmelt can all influence stormwater pollutant concentrations.

The annual pollutant load can have long-term effects on stream water quality, and is particularly important information for stormwater managers to have when dealing with non-point source pollution control. The Simple Method is a model developed to estimate the pollutant load for chemical pollutants, assuming that the annual pollutant load is a function of IC. It is an effective method for determining annual sediment, nutrient, and trace metal loads. It cannot always be applied to other stormwater pollutants, since they are not always correlated with IC.

The direct water quality impact of stormwater pollutants also depends on the type of pollutant, as different pollutants impact streams differently. For example, sediments affect stream habitat and aquatic biodiversity; nutrients cause eutrophication; metals, hydrocarbons, deicers, and MTBE can be toxic to aquatic life; and organic carbon can lower dissolved oxygen levels.

The impact stormwater pollutants have on water quality can also directly influence human uses and activities. Perhaps the pollutants of greatest concern are those with associated public health impacts, such as bacteria and pathogens. These pollutants can affect the availability of clean drinking water and limit consumptive recreational activities, such as swimming or fishing. In extreme situations, these pollutants can even limit contact recreational activities such as boating and wading.

It should be noted that although there is much research available on the effects of urbanization on water quality, the majority has not been focused on the impact on streams, but on the response of lakes, reservoirs, rivers and estuaries. It is also important to note that not all pollutants are equally represented in monitoring conducted to date. While we possess excellent monitoring data for sediment, nutrients and trace metals, we have relatively little monitoring data for pesticides, hydrocarbons, organic carbon, deicers, and MTBE.

4.2 Summary of National and Regional Stormwater Pollutant Concentration Data

4.2.1 National Data

National mean concentrations of typical stormwater pollutants are presented in Table 16. National stormwater data are compiled from the Nationwide Urban Runoff Program (NURP), with additional data obtained from the U.S.Geological Survey (USGS), as well as initial stormwater monitoring conducted for EPA's National Pollutant Discharge Elimination System (NPDES) Phase I stormwater program.

In most cases, stormwater pollutant data is reported as an event mean concentration (EMC), which represents the average concentration of the pollutant during an entire stormwater runoff event.

When evaluating stormwater EMC data, it is important to keep in mind that regional EMCs can differ sharply from the reported national pollutant EMCs. Differences in EMCs between regions are often attributed to the variation in the amount and frequency of rainfall and snowmelt.

4.2.2 Regional Differences Due to Rainfall

The frequency of rainfall is important, since it influences the accumulation of pollutants on IC that are subsequently available for wash-off during storm events. The USGS developed a national stormwater database encompassing 1,123 storms in 20 metropolitan areas and used it as the primary data source to define regional differences in stormwater EMCs. Driver (1988) performed regression analysis to determine which factors had the greatest influence on stormwater EMCs and determined that annual rainfall depth was the best overall predictor. Driver grouped together stormwater EMCs based on the depth of average annual rainfall, and Table 17 depicts the regional rainfall groupings and general trends for each region. Table 18 illustrates the distribution of stormwater EMCs for a range of rainfall regions from 13 local studies, based on other monitoring studies. In general, stormwater EMCs for nutrients, suspended sediment and metals tend to be higher in arid and semi-arid

Table 16: National EMCs for Stormwater Pollutants

Pollutant	Source	EMCs		Number of Events
		Mean	Median	
Sediments (mg/l)				
TSS	(1)	78.4	54.5	3047
Nutrients (mg/l)				
Total P	(1)	0.32	0.26	3094
Soluble P	(1)	0.13	0.10	1091
Total N	(1)	2.39	2.00	2016
TKN	(1)	1.73	1.47	2693
Nitrite & Nitrate	(1)	0.66	0.53	2016
Metals (Fg/l)				
Copper	(1)	13.4	11.1	1657
Lead	(1)	67.5	50.7	2713
Zinc	(1)	162	129	2234
Cadmium	(1)	0.7	N/R	150
Chromium	(4)	4	7	164
Hydrocarbons (mg/l)				
PAH	(5)	3.5	N/R	N/R
Oil and Grease	(6)	3	N/R	N/R
Bacteria and Pathogens (colonies/ 100ml)				
Fecal Coliform	(7)	15,038	N/R	34
Fecal Streptococci	(7)	35,351	N/R	17
Organic Carbon (mg/l)				
TOC	(11)	17	15.2	19 studies
BOD	(1)	14.1	11.5	1035
COD	(1)	52.8	44.7	2639
MTBE	(8)	N/R	1.6	592
Pesticides (Fg/l)				
Diazinon	(10)	N/R	0.025	326
	(2)	N/R	0.55	76
Chlorpyrifos	(10)	N/R	N/R	327
Atrazine	(10)	N/R	0.023	327
Prometon	(10)	N/R	0.031	327
Simazine	(10)	N/R	0.039	327
Chloride (mg/l)				
Chloride	(9)	N/R	397	282

Sources: [1] Smullen and Cave, 1998; [2] Brush et al., 1995; [3] Baird et al., 1996; [4] Bannerman et al., 1996; [5] Rabanal and Grizzard, 1995; [6] Crunkilton et al., 1996; [7] Schueler, 1999; [8] Delzer, 1996; [9] Environment Canada, 2001; [10] USEPA, 1998; [11] CWP, 2001a N/R - Not Reported

Table 17: Regional Groupings by Annual Rainfall Amount (Driver, 1988)

Region	Annual Rainfall	States Monitored	Concentration Data
Region I: Low Rainfall	<20 inches	AK, CA, CO, NM, UT	Highest mean and median values for Total N, Total P, TSS and COD
Region II: Moderate Rainfall	20 40 inches	HA, IL, MI, MN, MI, NY, TX, OR, OH, WA, WI	Higher mean and median values than Region III for TSS, dissolved phosphorus and cadmium
Region III: High Rainfall	>40 inches	FL, MD, MA, NC, NH, NY, TX, TN, AR	Lower values for many parameters likely due to the frequency of storms and the lack of build up in pollutants

regions and tend to decrease slightly when annual rainfall increases (Table 19).

It is also hypothesized that a greater amount of sediment is eroded from pervious surfaces in arid or semi-arid regions than in humid regions due to the sparsity of protective vegetative cover. Table 19 shows that the highest concentrations of total suspended solids were recorded in regions with least rainfall. In addition, the chronic toxicity standards for several metals are most frequently exceeded during low rainfall regions (Table 20).

4.2.3 Cold Region Snowmelt Data

In colder regions, snowmelt can have a significant impact on pollutant concentrations. Snow accumulation in winter coincides with pollutant build-up; therefore, greater concentrations of pollutants are measured during snowmelt events. Sources of snowpack pollution in urban areas include wet and dry atmospheric deposition, traffic emissions, urban litter, deteriorated infrastructure, and deicing chemicals and abrasives (WERF, 1999).

Oberts *et al.* (1989) measured snowmelt pollutants in Minnesota streams and found that as much as 50% of annual sediment, nutrient, hydrocarbon and metal loads could be attributed to snowmelt runoff during late winter and early spring. This trend probably applies to any region where snow cover persists through much of the winter. Pollutants accumulate in the snowpack and then contribute high concentrations during snowmelt runoff. Oberts (1994) described four types of snowmelt runoff events and the resulting pollutant characteristics (Table 21).

A typical hydrograph for winter and early spring snow melts in a northern cold climate is portrayed in Figure 29. The importance of snowpack melt on peak runoff during March 1989 can clearly be seen for an urban watershed located in St. Paul, Minnesota.

Major source areas for snowmelt pollutants include snow dumps and roadside snowpacks. Pollutant concentrations in snow dumps can be as much as five times greater than typical stormwater pollutant concentrations (Environment Canada, 2001). Snow dumps and packs accumulate pollutants over the winter months and can release them during a few rain or snow melt events in the early spring. High levels of chloride, lead, phosphorus, biochemical oxygen demand, and total suspended solids have been reported in snow pack runoff (La Barre *et al*, 1973; Oliver *et al.*, 1974; Pierstorff and Bishop, 1980; Scott and Wylie, 1980; Van Loon, 1972).

Atmospheric deposition can add pollutants to snow piles and snowpacks. Deposited pollutants include trace metals, nutrients and particles that are primarily generated by fossil fuel combustion and industrial emissions (Boom and Marsalek, 1988; Horkeby and Malmqvist, 1977; Malmqvist, 1978; Novotny and Chester, 1981; Schrimpff and Herrman, 1979).

Table 18: Stormwater Pollutant Event Mean Concentration for Different U.S. Regions
(Units: mg/l, except for metals which are in *Fg/l*)

		Region I - Low Rainfall				Region II - Moderate Rainfall			Region III - High Rainfall				Snow
	National	Phoenix, AZ	San Diego, CA	Boise, ID	Denver, CO	Dallas, TX	Marquette, MI	Austin, TX	MD	Louisville, KY	GA	FL	MN
Reference	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(11)	(12)
Annual Rainfall (in.)	N/A	7.1"	10"	11"	15"	28"	32"	32"	41"	43"	51"	52"	N/R
Number of Events	3000	40	36	15	35	32	12	N/R	107	21	81	N/R	49
Pollutant													
TSS	78.4	227	330	116	242	663	159	190	67	98	258	43	112
Total N	2.39	3.26	4.55	4.13	4.06	2.70	1.87	2.35	N/R	2.37	2.52	1.74	4.30
Total P	0.32	0.41	0.7	0.75	0.65	0.78	0.29	0.32	0.33	0.32	0.33	0.38	0.70
Soluble P	0.13	0.17	0.4	0.47	N/R	N/R	0.04	0.24	N/R	0.21	0.14	0.23	0.18
Copper	14	47	25	34	60	40	22	16	18	15	32	1.4	N/R
Lead	68	72	44	46	250	330	49	38	12.5	60	28	8.5	100
Zinc	162	204	180	342	350	540	111	190	143	190	148	55	N/R
BOD	14.1	109	21	89	N/R	112	15.4	14	14.4	88	14	11	N/R
COD	52.8	239	105	261	227	106	66	98	N/R	38	73	64	112

Sources: Adapted from Caraco, 2000a: [(1)] Smullen and Cave, 1998; [(2)] Lopes et al.; 1995; [(3)] Schiff, 1996; [(4)] Kjelstrom, 1995 (computed); [(5)] DRCOG, 1983, [(6)] Brush et al.*, 1995; [(7)] Steuer* et al.*, 1997; [(8)] Barrett* et al.*, 1995; [(9)] Barr, 1997; [(10)] Evaldi* et al.*, 1992; [(11)] Thomas and McClelland, 1995; [(12)] Oberts, 1994 N/R = Not Reported; N/A = Not Applicable*

Table 19: Mean and Median Nutrient and Sediment Stormwater Concentrations for Residential Land Use Based on Rainfall Regions (Driver, 1988)

Region	Total N (median)	Total P (median)	TSS (mean)
Region I: Low Rainfall	4	0.45	320
Region II: Moderate Rainfall	2.3	0.31	250
Region III: High Rainfall	2.15	0.31	120

Table 20: EPA 1986 Water Quality Standards and Percentage of Metal Concentrations Exceeding Water Quality Standards by Rainfall Region (Driver, 1988)				
	Cadmium	**Copper**	**Lead**	**Zinc**
EPA Standards	10 Fg/l	12 Fg/l	32 Fg/l	47 Fg/l
	Percent Exceedance of EPA Standards			
Region I: Low Rainfall	1.5%	89%	97%	97%
Region II: Moderate Rainfall	0	78%	89%	85%
Region III: High Rainfall	0	75%	91%	84%

Table 21: Runoff and Pollutant Characteristics of Snowmelt Stages (Oberts, 1994)			
Snowmelt Stage	Duration /Frequency	Runoff Volume	Pollutant Characteristics
Pavement	Short, but many times in winter	Low	Acidic, high concentrations of soluble pollutants; Chloride, nitrate, lead; total load is minimal
Roadside	Moderate	Moderate	Moderate concentrations of both soluble and particulate pollutants
Pervious Area	Gradual, often most at end of season	High	Dilute concentrations of soluble pollutants; moderate to high concentrations of particulate pollutants depending on flow
Rain-on-Snow	Short	Extreme	High concentrations of particulate pollutants; moderate to high concentrations of soluble pollutants; high total load

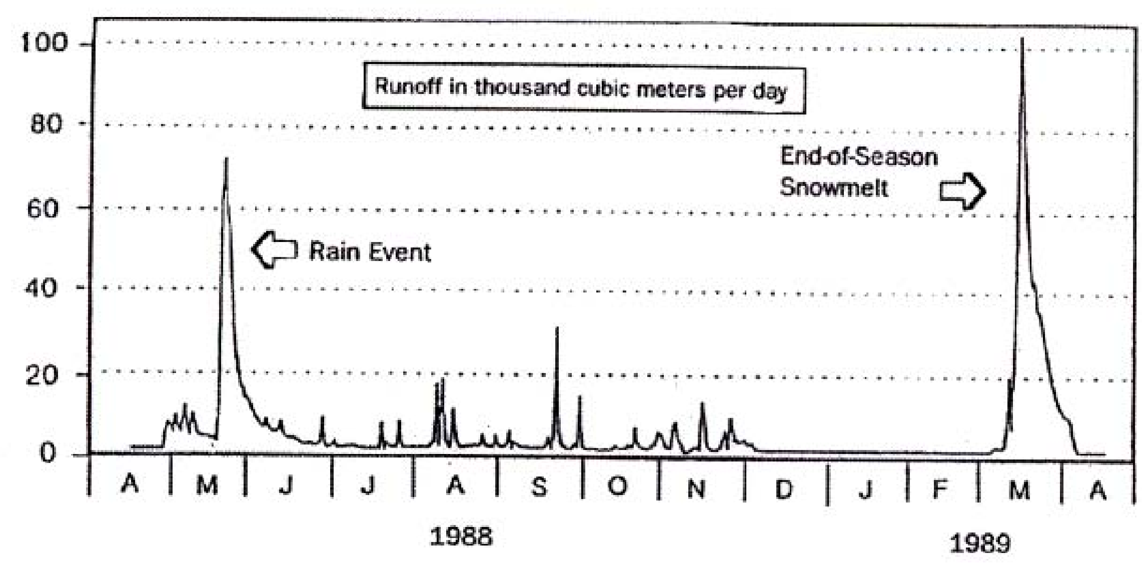


Figure 29: Snowmelt Runoff Hydrograph for Minneapolis Stream (Oberts, 1994)

4.3 Relationship Between Pollutant Loads and IC: The Simple Method

Urban stormwater runoff contains a wide range of pollutants that can degrade downstream water quality. The majority of stormwater monitoring research conducted to date supports several generalizations. First, the unit area pollutant load delivered to receiving waters by stormwater runoff increases in direct proportion to watershed IC. This is not altogether surprising, since pollutant load is the product of the average pollutant concentration and stormwater runoff volume. Given that runoff volume increases in direct proportion to IC, pollutant loads must automatically increase when IC increases, as long the average pollutant concentration stays the same (or increases). This relationship is a central assumption in most simple and complex pollutant loading models (Bicknell *et al.*, 1993; Donigian and Huber, 1991; Haith *et al.*, 1992; Novotny and Chester, 1981; NVPDC, 1987; Pitt and Voorhees, 1989).

Recognizing the relationship between IC and pollutant loads, Schueler (1987) developed the "Simple Method" to quickly and easily estimate stormwater pollutant loads for small urban watersheds (see Figure 30). Estimates of pollutant loads are important to watershed managers as they grapple with costly decisions on non-point source control. The Simple Method is empirical in nature and utilizes the extensive regional and national database (Driscoll, 1983; MWCOG, 1983; USEPA, 1983). Figure 30 provides the basic equations to estimate pollutant loads using the Simple

Figure 30: The Simple Method - Basic Equations

The Simple Method estimates pollutant loads as the product of annual runoff volume and pollutant EMC, as:

$$(1)\ L = 0.226 * R * C * A$$

Where: L = Annual load (lbs), and:
R = Annual runoff (inches)
C = Pollutant concentration in stormwater, EMC (mg/l)
A = Area (acres)
0.226 = Unit conversion factor

For bacteria, the equation is slightly different, to account for the differences in units. The modified equation for bacteria is:

$$(2)\ L = 1.03 * 10^{-3} * R * C * A$$

Where: L = Annual load (Billion Colonies), and:
R = Annual runoff (inches)
C = Bacteria concentration (#/100 ml)
A = Area (acres)
$1.03 * 10^{-3}$ = Unit conversion factor

Annual Runoff

The Simple Method calculates the depth of annual runoff as a product of annual runoff volume and a runoff coefficient (Rv). Runoff volume is calculated as:

$$(3)\ R = P * P_j * Rv$$

Where: R = Annual runoff (inches), and:
P = Annual rainfall (inches)
P_j = Fraction of annual rainfall events that produce runoff (usually 0.9)
Rv = Runoff coefficient

In the Simple Method, the runoff coefficient is calculated based on IC in the subwatershed. The following equation represents the best fit line for the data set (N=47, R^2=0.71).

$$(4)\ Rv=0.05+0.9Ia$$

Where: Rv = runoff coefficient, and:
Ia = Impervious fraction

Method. It assumes that loads of stormwater pollutants are a direct function of watershed IC, as IC is the key independent variable in the equation.

The technique requires a modest amount of information, including the subwatershed drainage area, IC, stormwater runoff pollutant EMCs, and annual precipitation. With the Simple Method, the investigator can either divide up land use into specific areas (i.e. residential, commercial, industrial, and roadway) and calculate annual pollutant loads for each land use, or utilize a generic urban land use. Stormwater pollutant EMC data can be derived from the many summary tables of local, regional, or national monitoring efforts provided in this chapter (e.g., Tables 16, 18, 22, 28, 30, 35, 36, 40, and 44). The model also requires different IC values for separate land uses within a subwatershed. Representative IC data from Cappiella and Brown (2001) were provided in Table 2 (Chapter 1).

Additionally, the Simple Method should not be used to estimate annual pollutant loads of deicers, hydrocarbons and MTBE, because they have not been found to be correlated with IC. These pollutants have been linked to other indicators. Chlorides, hydrocarbons and MTBE are often associated with road density and vehicle miles traveled (VMT). Pesticides are associated with turf area, and traffic patterns and "hotspots" have been noted as potential indicators for hydrocarbons and MTBE.

Limitations of the Simple Method

The Simple Method should provide reasonable estimates of changes in pollutant export resulting from urban development. However, several caveats should be kept in mind when applying this method.

The Simple Method is most appropriate for assessing and comparing the relative stormflow pollutant load changes from different land uses and stormwater treatment scenarios. The Simple Method provides estimates of storm pollutant export that are probably close to the "true" but unknown value for a development site, catchment, or subwatershed. However, it is very important not to overemphasize the precision of the load estimate obtained. For example, it would be inappropriate to use the Simple Method to evaluate relatively similar development scenarios (e.g., 34.3% versus 36.9% IC). The Simple Method provides a general planning estimate of likely storm pollutant export from areas at the scale of a development site, catchment or subwatershed. More sophisticated modeling is needed to analyze larger and more complex watersheds.

In addition, the Simple Method only estimates pollutant loads generated during storm events. It does not consider pollutants associated with baseflow during dry weather. Typically, baseflow is negligible or non-existent at the scale of a single development site and can be safely neglected. However, catchments and subwatersheds do generate significant baseflow volume. Pollutant loads in baseflow are generally low and can seldom be distinguished from natural background levels (NVPDC, 1979).

Consequently, baseflow pollutant loads normally constitute only a small fraction of the total pollutant load delivered from an urban area. Nevertheless, it is important to remember that the load estimates refer only to storm event derived loads and should not be confused with the total pollutant load from an area. This is particularly important when the development density of an area is low. For example, in a low density residential subwatershed (IC < 5%), as much as 75% of the annual runoff volume could occur as baseflow. In such a case, annual baseflow load may be equivalent to the annual stormflow load.

4.4 Sediment

Sediment is an important and ubiquitous pollutant in urban stormwater runoff. Sediment can be measured in three distinct ways: Total Suspended Solids (TSS), Total Dissolved Solids (TDS) and turbidity. TSS is a measure of the total mass suspended sediment particles in water. The measurement of TSS in urban stormwater helps to estimate sediment load transported to local and downstream receiving waters. Table 22 summarizes stormwater EMCs for total suspended solids, as reported by Barrett *et al.* (1995), Smullen and Cave (1998), and USEPA (1983). TDS is a measure of the dissolved solids and minerals present in stormwater runoff and is used as a primary indication of the purity of drinking water. Since few stormwater monitoring efforts have focused on TDS, they are not reported in this document. Turbidity is a measure of how suspended solids present in water reduce the ability of light to penetrate the water column. Turbidity can exert impacts on aquatic biota, such as the ability of submerged aquatic vegetation to receive light and the ability of fish and aquatic insects to use their gills (Table 23).

4.4.1 Concentrations

TSS concentrations in stormwater across the country are well documented. Table 18 reviews mean TSS EMCs from 13 communities across the country and reveals a wide range of recorded concentrations. The lowest concentration of 43 mg/l was reported in Florida, while TSS reached 663 mg/l in Dallas, Texas. Variation in sediment concentrations has been attributed to regional rainfall differences (Driver, 1988); construction site runoff (Leopold, 1968); and bank erosion (Dartiguenave *et al.*, 1997). National values are provided in Table 22.

Turbidity levels are not as frequently reported in national and regional monitoring summaries. Barrett and Malina (1998) monitored turbidity at two sites in Austin, Texas and reported a mean turbidity of 53 NTU over 34 storm events (Table 22).

4.4.2 Impacts of Sediment on Streams

The impacts of sediment on aquatic biota are well documented and can be divided into impacts caused by suspended sediment and those caused by deposited sediments (Tables 23 and 24).

In general, high levels of TSS and/or turbidity can affect stream habitat and cause sedimentation in downstream receiving waters. Deposited sediment can cover benthic organisms such as aquatic insects and freshwater mussels. Other problems associated with high sediments loads include stream warming by reflecting radiant energy due to increased turbidity (Kundell and Rasmussen, 1995), decreased flow capacity (Leopold, 1973), and increasing overbank flows (Barrett and Malina, 1998). Sediments also transport other pollutants which bind to sediment particles. Significant levels of pollutants can be transported by sediment during stormwater runoff events,

Table 22: EMCs for Total Suspended Solids and Turbidity

Pollutant	EMCs		Number of Events	Source
	Mean	Median		
TSS (mg/l)	78.4	54.5	3047	Smullen and Cave, 1998
	174	113	2000	USEPA, 1983
Turbidity (NTU)	53	N/R	423	Barrett and Malina, 1998

N/R = Not Reported

Table 23: Summary of Impacts of Suspended Sediment on the Aquatic Environment (Schueler and Holland, 2000)
Abrades and damages fish gills, increasing risk of infection and disease
Scouring of periphyton from stream (plants attached to rocks)
Loss of sensitive or threatened fish species when turbidity exceeds 25 NTU
Shifts in fish community toward more sediment-tolerant species
Decline in sunfish, bass, chub and catfish when month turbidity exceeds 100 NTU
Reduces sight distance for trout, with reduction in feeding efficiency
Reduces light penetration causing reduction in plankton and aquatic plant growth
Adversely impacts aquatic insects, which are the base of the food chain
Slightly increases the stream temperature in the summer
Suspended sediments can be a major carrier of nutrients and metals
Reduces anglers chances of catching fish

Table 24: Summary of Impacts of Deposited Sediments on the Aquatic Environment (Schueler and Holland, 2000)
1. Physical smothering of benthic aquatic insect community
2. Reduced survival rates for fish eggs
3. Destruction of fish spawning areas and eggs
4. Embeddedness of stream bottom reduced fish and macroinvertebrate habitat value
5. Loss of trout habitat when fine sediments are deposited in spawning or riffle-runs
6. Sensitive or threatened darters and dace may be eliminated from fish community
7. Increase in sediment oxygen demand can deplete dissolved oxygen in streams
8. Significant contributing factor in the alarming decline of freshwater mussels
9. Reduced channel capacity, exacerbating downstream bank erosion and flooding
10. Reduced flood transport capacity under bridges and through culverts
11. Deposits diminish scenic and recreational values of waterways

including trace metals, hydrocarbons and nutrients (Crunkilton *et al.,* 1996; Dartiguenave *et al*., 1997; Gavin and Moore, 1982; Novotny and Chester, 1989; Schueler 1994b).

4.4.3 Sources and Source Areas of Sediment

Sediment sources in urban watersheds include stream bank erosion; erosion from exposed soils, such as from construction sites; and washoff from impervious areas (Table 25).

As noted in this chapter, streambank erosion is generally considered to be the primary source of sediment to urban streams. Recent studies by Dartiguenave *et al.* (1997) and Trimble (1997) determined that streambank erosion contributes the majority of the annual sediment budget of urban streams. Trimble (1997) directly measured stream cross sections, sediment aggradation and suspended sediment loads and determined that two-thirds of the annual sediment budget of a San Diego, California watershed was supplied by streambank erosion. Dartiguenave *et al.* (1997) developed a GIS based model in Austin, Texas to determine the effects of stream bank erosion on the annual sediment budget. They compared modeled sediment loads from the watershed with the actual sediment loads measured at USGS gaging stations and concluded that more than 75% of the sediment load came from streambank erosion. Dartiguenave *et al.* (1997) reported that sediment load per unit area increases with increasing IC (Figure 31).

Sediment loads are also produced by washoff of sediment particles from impervious areas and their subsequent transport in stormwater runoff sediment. Source areas include parking lots, streets, rooftops, driveways and lawns. Streets and parking lots build up dirt and grime from the wearing of the street surface, exhaust particulates, "blown on" soil and organic matter, and atmospheric deposition. Lawn runoff primarily contains soil and organic matter. Urban source areas that produce the highest TSS concentrations include streets, parking lots and lawns (Table 26).

Parking lots and streets are not only responsible for high concentrations of sediment but also high runoff volumes. The SLAMM source loading model (Pitt and Voorhees, 1989) looks at runoff volume and concentrations of pollutants from different urban land uses and predicts stream loading. When used in the Wisconsin and Michigan subwatersheds, it demonstrated that parking lots and streets were responsible for over 70% of the TSS delivered to the stream. (Steuer *et al*., 1997; Waschbusch *et al*., 2000).

Table 25: Sources and Loading of Suspended Solids Sediment in Urban Areas

Sources	Loading	Source
Bank Erosion	75% of stream sediment budget	Dartinguenave *et al*., 1997
	66% of stream sediment budget	Trimble, 1997
Overland Flow- Lawns	397 mg/l (geometric mean)	Bannerman *et al*., 1993
	262 mg/l	Steuer *et al*., 1997
	11.5% (estimated; 2 sites)	Waschbusch *et al*., 2000
Construction Sites	200 to 1200 mg/l	Table 27
Washoff from Impervious Surfaces	78 mg/l (mean)	Table 16

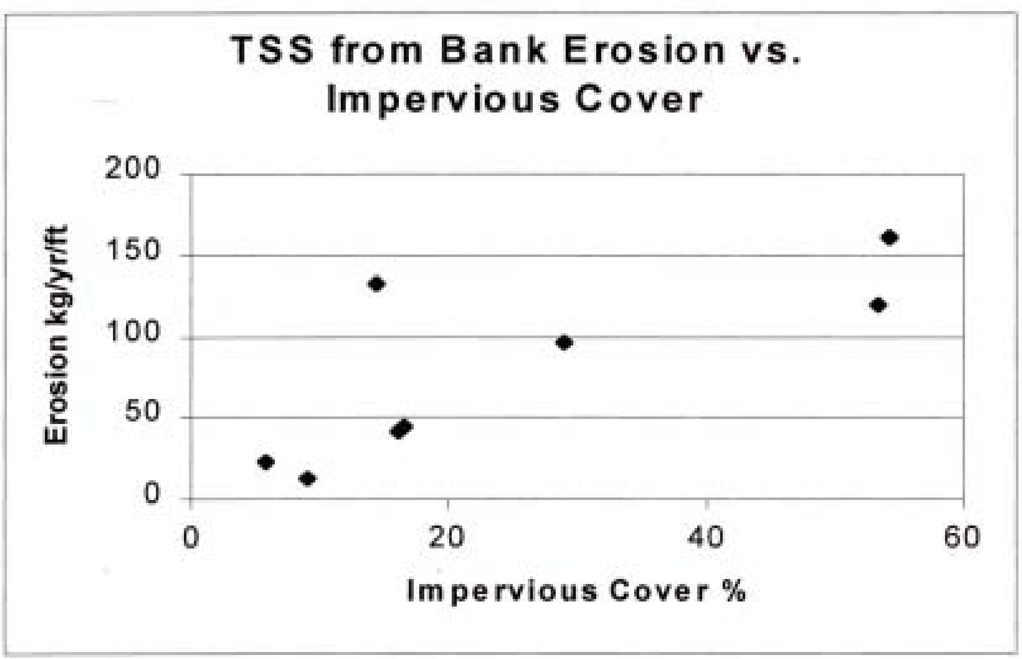


Figure 31: TSS from Bank Erosion vs. IC in Texas Streams (Daringuenave *et al*., 1997)

The third major source of sediment loads is erosion from construction sites. Several studies have reported extremely high TSS concentrations in construction site runoff, and these findings are summarized in Table 27. TSS concentrations from uncontrolled construction sites can be more than 150 times greater than those from undeveloped land (Leopold, 1968) and can be reduced if erosion and sediment control practices are applied to construction sites.

Table 26: Source Area Geometric Mean Concentrations for Suspended Solids in Urban Areas

Source Area	Suspended Solids (mg/l)		
Source	(1)	(2)	(3)
Commercial Parking Lot	110	58	51
High Traffic Street	226	232	65
Medium Traffic Street	305	326	51
Low Traffic Street	175	662	68
Commercial Rooftop	24	15	18
Residential Rooftop	36	27	15
Residential Driveway	157	173	N/R
Residential Lawn	262	397	59

Sources: (1) Steuer et al., 1997; (2) Bannerman et al., 1993; (3) Waschbusch et al., 2000; N/R = Not Reported

Table 27: Mean TSS Inflow and Outflow at Uncontrolled, Controlled and Simulated Construction Sites

Source	Mean Inflow TSS Concentration (mg/l)	Mean Outflow TSS Concentration (mg/l)	Location
Uncontrolled Sites			
Horner *et al.*, 1990	7,363	281	PNW
Schueler and Lugbill,1990	3,646	501	MD
York and Herb, 1978	4,200	N/R	MD
Islam *et al.*, 1988	2,950	N/R	OH
Controlled Sites			
Schueler and Lugbill, 1990	466	212	MD
Simulated Sediment Concentrations			
Jarrett, 1996	9,700	800	PA
Sturm and Kirby, 1991	1,500-4,500	200-1,000	GA
Barfield and Clar, 1985	1,000-5,000	200-1,200	MD
Dartiguenave *et al.*, 1997	N/R	600	TX

N/R = Not Reported

4.5 Nutrients

Nitrogen and phosphorus are essential nutrients for aquatic systems. However, when they appear in excess concentrations, they can exert a negative impact on receiving waters. Nutrient concentrations are reported in several ways. Nitrogen is often reported as nitrate (NO_3) and nitrite (NO_2), which are inorganic forms of nitrogen; total nitrogen (Total N), which is the sum of nitrate, nitrite, organic nitrogen and ammonia; and total Kjeldhal nitrogen (TKN), which is organic nitrogen plus ammonia.

Phosphates are frequently reported as soluble phosphorus, which is the dissolved and reactive form of phosphorus that is available for uptake by plants and animals. Total phosphorus (Total P) is also measured, which includes both organic and inorganic forms of phosphorus. Organic phosphorus is derived from living plants and animals, while inorganic phosphate is comprised of phosphate ions that are often bound to sediments.

4.5.1 Concentrations

Many studies have indicated that nutrient concentrations are linked to land use type, with urban and agricultural watersheds producing the highest nutrient loads (Chessman *et al.* 1992; Paul *et al.*, 2001; USGS, 2001b and Wernick *et al.*,1998). Typical nitrogen and phosphorus EMC data in urban stormwater runoff are summarized in Table 28.

Some indication of the typical concentrations of nitrate and phosphorus in stormwater runoff are evident in Figures 32 and 33. These graphs profile average EMCs in stormwater runoff recorded at 37 residential catchments across the U.S. The average nitrate EMC is remarkably consistent among residential neighborhoods, with most clustered around the mean of 0.6 mg/l and a range of 0.25 to 1.4 mg/l. The concentration of phosphorus during storms is also very consistent with a mean of 0.30 mg/l and a rather tight range of 0.1 to 0.66 mg/l (Schueler, 1995).

The amount of annual rainfall can also influence the magnitude of nutrient concentrations in stormwater runoff. For example, both Caraco (2000a) and Driver (1988) reported that the highest nutrient EMCs were found in stormwater from arid or semi-arid regions.

Table 28: EMCs of Phosphorus and Nitrogen Urban Stormwater Pollutants

Pollutant	EMCs (mg/l)		Number of Events	Source
	Mean	Median		
Total P	0.315	0.259	3094	Smullen and Cave, 1998
	0.337	0.266	1902	USEPA, 1983
Soluble P	0.129	0.103	1091	Smullen and Cave, 1998
	0.1	0.078	767	USEPA, 1983
Total N	2.39	2.00	2016	Smullen and Cave, 1998
	2.51	2.08	1234	USEPA, 1983
TKN	1.73	1.47	2693	Smullen and Cave, 1998
	1.67	1.41	1601	USEPA, 1983
Nitrite & Nitrate	0.658	0.533	2016	Smullen and Cave, 1998
	0.837	0.666	1234	USEPA, 1983

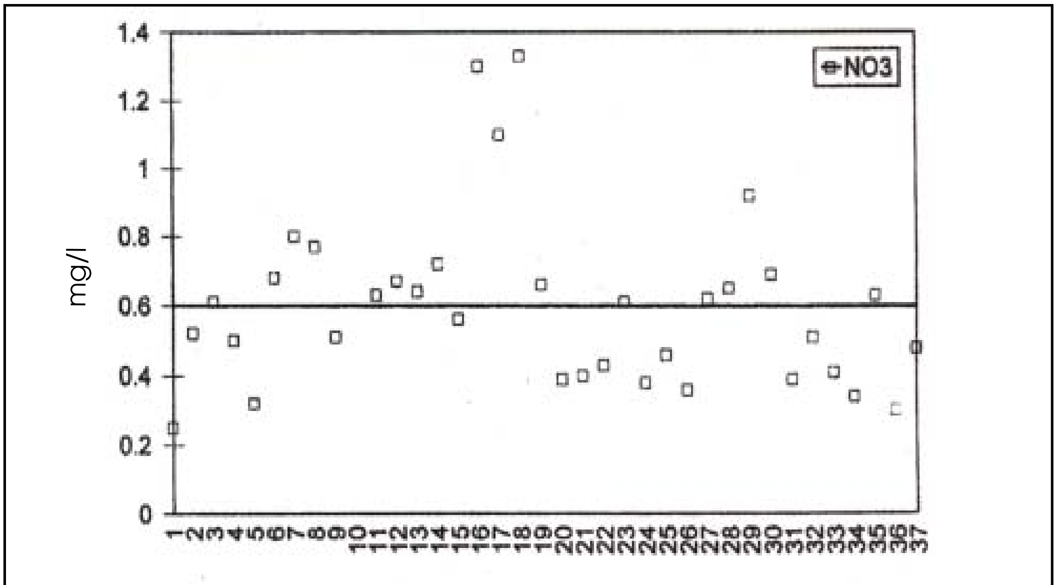


Figure 32: Nitrate-Nitrogen Concentration in Stormwater Runoff at 37 Sites Nationally (Schueler, 1999)

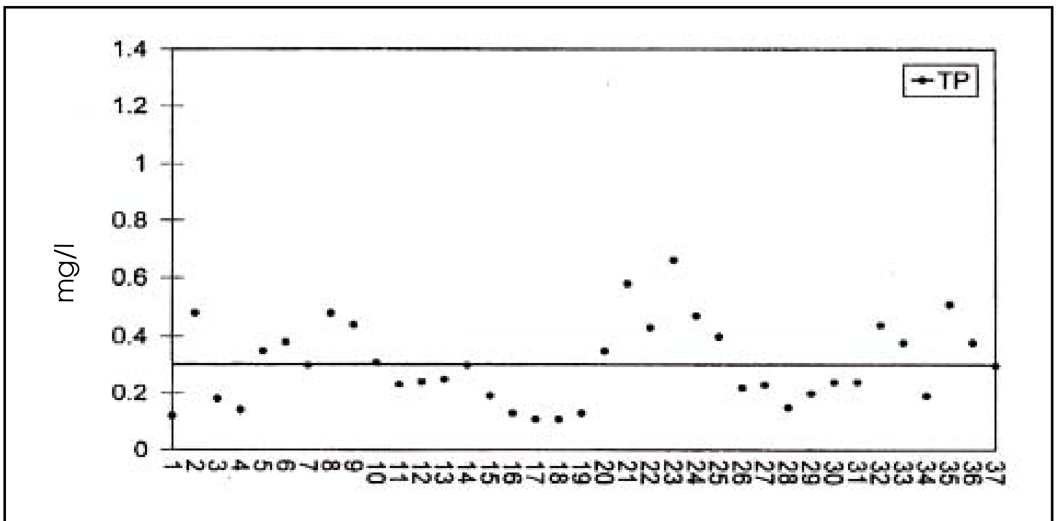


Figure 33: Total Phosphorus Concentration in Stormwater at 37 Sites Nationally (Schueler, 1999)

4.5.2 Impacts of Nutrients on Streams

Much research on the impact of nutrient loads has been focused on lakes, reservoirs and estuaries, which can experience eutrophication. Nitrogen and phosphorus can contribute to algae growth and eutrophic conditions, depending on which nutrient limits growth (USEPA, 1998). Dissolved oxygen is also affected by eutrophication. When algae or aquatic plants that are stimulated by excess nutrients die off, they are broken down by bacteria, which depletes the oxygen in the water. Relatively few studies have specifically explored the impact of nutrient enrichment on urban streams. Chessman *et al*. (1992) studied the limiting nutrients for periphyton growth in a variety of streams and noted that the severity of eutrophication was related to low flow conditions. Higher flow rates in streams may cycle nutrients faster than in slow flow rates, thus diminishing the extent of stream eutrophication.

4.5.3 Sources and Source Areas of Nutrients

Phosphorus is normally transported in surface water attached to sediment particles or in soluble forms. Nitrogen is normally transported by surface water runoff in urban watersheds. Sources for nitrogen and phosphorus in urban stormwater include fertilizer, pet waste, organic matter (such as leaves and detritus), and stream bank erosion. Another significant source of nutrients is atmospheric deposition. Fossil fuel combustion by automobiles, power plants and industry can supply nutrients in both wet fall and dry fall. The Metropolitan Washington Council of Governments (MWCOG, 1983) estimated total annual atmospheric deposition rates of 17 lbs/ac for nitrogen and 0.7 lbs/ac for phosphorus in the Washington, D.C. metro area.

Research from the upper Midwest suggests "hot spot" sources can exist for both nitrogen and phosphorus in urban watersheds. Lawns, in particular, contribute greater concentrations of Total N, Total P and dissolved phosphorus than other urban source areas. Indeed, source research suggests that nutrient concentrations in lawn runoff can be as much as four times greater than other urban sources such as streets, rooftops or driveways (Bannerman *et al.*, 1993; Steuer *et al.*, 1997 and Waschbusch *et al.*, 2000) (Table 29). This finding is significant, since lawns can comprise more than 50% of the total area in suburban watersheds. Lawn care, however, has seldom been directly linked to elevated nutrient concentrations during storms. A very recent lakeshore study noted that phosphorus concentrations were higher in fertilized lawns compared to unfertilized lawns, but no significant difference was noted for nitrogen (Garn, 2002).

Wash-off of deposited nutrients from IC is thought to be a major source of nitrogen and phosphorus during storms (MWCOG, 1983). While the concentration of nitrogen and phosphorus from parking lots and streets is lower than lawns, the volume of runoff is significantly higher. In two studies using the SLAMM source loading model (Pitt and Voorhees, 1989), parking lots and streets were responsible for over 30% of the nitrogen and were second behind lawns in their contributions to the phosphorus load (Steuer *et al.*, 1997; Waschbusch *et al.*, 2000).

Table 29: Source Area Monitoring Data for Total Nitrogen and Total Phosphorous in Urban Areas

Source Area	Total N (mg/l)	Total P (mg/l)		
Source	(1)	(1)	(2)	(3)
Commercial Parking Lot	1.94	0.20	N/R	0.10
High Traffic Street	2.95	0.31	0.47	0.18
Med. Traffic Street	1.62	0.23	1.07	0.22
Low Traffic Street	1.17	0.14	1.31	0.40
Commercial Rooftop	2.09	0.09	0.20	0.13
Residential Rooftop	1.46	0.06	0.15	0.07
Residential Driveway	2.10	0.35	1.16	N/R
Residential Lawn	9.70	2.33	2.67	0.79
Basin Outlet	1.87	0.29	0.66	N/R

(1) Steuer et al., 1997; (2) Bannerman et al., 1993; (3) Waschbusch et al., 2000; N/R= Not Reported

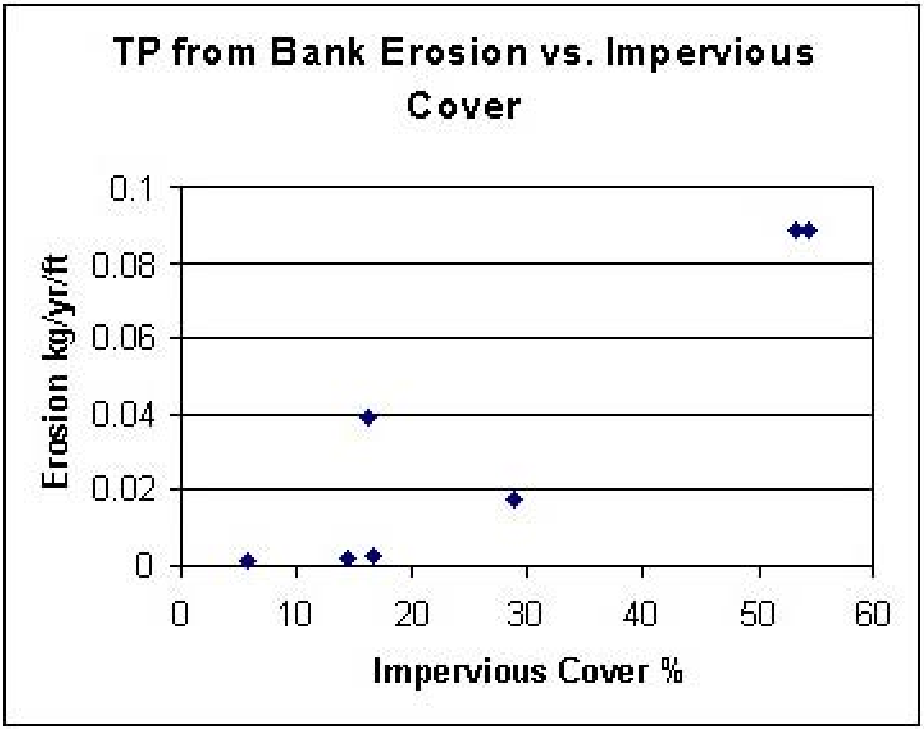


Figure 34: Total Phosphorus from Bank Erosion as a Function of IC in Texas Streams (Dartiguenave *et al.*, 1997)

Streambank erosion also appears to be a major source of nitrogen and phosphorus in urban streams. Both nitrogen and phosphorus are often attached to eroded bank sediment, as indicated in a recent study by Dartiguenave *et al.* (1997) in Austin, Texas. They showed that channel erosion contributed nearly 50% of the Total P load shown for subwatersheds with IC levels between 10 and 60 % (Figure 34). These findings suggest that prevention or reduction of downstream channel erosion may be an important nutrient reduction strategy for urban watersheds.

Snowmelt runoff generally has higher nutrient EMCs, compared to stormwater runoff. Oberts (1994) found that TKN and nitrate EMCs were much higher in snowmelt at all sites. The same pattern has also been observed for phosphorus EMCs during snowmelt and stormwater runoff. Zapf-Gilje *et al*. (1986) found that the first 20% of snowmelt events contained 65% of the phosphorus and 90% of the nitrogen load. Ayers *et al.* (1985) reported that a higher percentage of the annual nitrate, TKN and phosphorus load was derived from snowmelt runoff compared to stormwater runoff in an urban Minnesota watershed, which presumably reflects the accumulation of nutrients in the snowpack during the winter.

4.6 Trace Metals

Many trace metals can be found at potentially harmful concentrations in urban stormwater. Certain metals, such as zinc, copper, lead, cadmium and chromium, are consistently present at concentrations that may be of concern. These metals primarily result from the use of motor vehicles, weathering of metals and paints, burning of fossil fuels and atmospheric deposition.

Metals are routinely reported as the total recoverable form or the dissolved form. The dissolved form refers to the amount of metal dissolved in the water, which excludes metals attached to suspended particles that cannot pass through a 0.45 micron filter. Total recoverable refers to the concentration of an unfiltered sample that is treated with hot dilute mineral acid. In general, the toxicity of metals is related more to the dissolved form than the recoverable form.

4.6.1 Concentrations

Stormwater EMCs for zinc, copper, lead, cadmium and chromium vary regionally and are reviewed in Table 30. Regional differences in trace metal concentrations and water quality standard exceedence appears to be related to climate. In general, drier regions often have a

Table 30: EMCs and Detection Frequency for Metals in Urban Stormwater

Metal	Detection Frequency[1]	EMCs (Fg/l)		Number of Events	Source
		Mean	Median		
Zinc	94%	162	129	2234	Smullen and Cave, 1998
		176	140	1281	USEPA, 1983
Copper	91%	13.5	11.1	1657	Smullen and Cave, 1998
		66.6	54.8	849	USEPA, 1983
Lead	94%	67.5	50.7	2713	Smullen and Cave, 1998
		175[2]	131[2]	1579	USEPA, 1983
Cadmium	48%	0.7	N/R	150	USEPA, 1983
		0.5	N/R	100	USEPA, 1993
		N/R	0.75 **R** 0.96 **C** 2.1 **I**	30	Baird *et al.*, 1996
		3 **I** 1**U**	N/R	9	Doerfer and Urbonas, 1993
Chromium	58%	4	N/R	32	Baird *et al.*, 1996
		N/R	2.1 **R** 10 **C** 7 **I**	30	Baird *et al.*, 1996
		N/R	7	164	Bannerman *et al.*, 1993

N/R = Not Reported; R- Residential, C- Commercial, I- Industrial; (1) as reprinted in USEPA, 1983; (2) Lead levels have declined over time with the introduction of unleaded gasoline

Table 31: Average Total Recoverable and Dissolved Metals for 13 Stormwater Flows and Nine Baseflow Samples from Lincoln Creek in 1994 (Crunkilton *et al.*, 1996)

	Total Recoverable		Dissolved	
Metal (Fg/l)	**Storm Flow**	**Baseflow**	**Storm Flow**	**Baseflow**
Lead	35	3	1.7	1.2
Zinc	133	22	13	8
Copper	23	7	5	4
Cadmium	0.6	0.1	0.1	0.1

higher risk of exceeding trace metal concentration standards.

Crunkilton *et al.* (1996) measured recoverable and dissolved metals concentrations in Lincoln Creek, Wisconsin and found higher EMCs during storm events compared to baseflow periods (Table 31). They also found that total recoverable metal concentrations were almost always higher than the dissolved concentration (which is the more available form).

4.6.2 Impacts of Trace Metals on Streams

Although a great deal is known about the concentration of metals in urban stormwater, much less is known about their possible toxicity on aquatic biota. The primary concern related to the presence of trace metals in streams is their potential toxicity to aquatic organisms. High concentrations can lead to bioaccumulation of metals in plants and animals, possible chronic or acute toxicity, and contamination of sediments, which can affect bottom dwelling organisms (Masterson and Bannerman, 1994). Generally, trace metal concentrations found in urban stormwater are not high enough to cause acute toxicity (Field and Pitt, 1990). The cumulative accumulation of trace metal concentrations in bottom sediments and animal tissues are of greater concern. Some evidence exists for trace metal accumulation in bottom sediments of receiving waters and for bioaccumulation in aquatic species (Bay and Brown, 2000 and Livingston, 1996).

Relatively few studies have examined the chronic toxicity issue. Crunkilton *et al.* (1996) found that concentrations of lead, zinc and copper exceeded EPA's Chronic Toxicity Criteria more than 75% of the time in stormflow in stormwater samples for Lincoln Creek in Wisconsin. When exposed to storm and base flows in Lincoln Creek, *Ceriodaphnia dubia*, a common invertebrate test species, demonstrated significant mortality in extended flow-through tests. Around 30% mortality was recorded after seven days of exposure and 70% mortality was recorded after 14 days.

Crunkilton *et al.* (1996) also found that significant mortality in bullhead minnows occurred in only 14% of the tests by the end of 14 days, but mortality increased to 100% during exposures of 17 to 61 days (see Table 32). In a related study in the same watershed, Masterson and Bannerman (1994) determined that crayfish in Lincoln Creek had elevated levels of lead, cadmium, chromium and copper when compared to crayfish from a reference stream. The Lincoln Creek research provides limited evidence that prolonged exposure to trace metals in urban streams may result in significant toxicity.

Most toxicity research conducted on urban stormwater has tested for acute toxicity over a short period of time (two to seven days). Shorter term whole effluent toxicity protocols are generally limited to seven days (Crunkilton *et al.*, 1996). Research by Ellis (1986) reported delayed toxicity in urban streams. Field and Pitt (1990) demonstrated that pollutants deposited to the stream during storm events may take upwards of 10 to 14 days to exert influence. The research suggests that longer term in-situ and flow-through monitoring are needed to definitively answer the question whether metal levels in stormwater can be chronically toxic.

An additional concern is that trace metals co-occur with other pollutants found in urban stormwater, and it is not clear whether they interact to increase or decrease potential toxicity. Hall and Anderson (1988) investigated the toxicity and chemical composition of urban stormwater runoff in British Columbia and found that the interaction of pollutants changed the toxicity of some metals. In laboratory analysis with *Daphnia pulex*, an aquatic invertebrate, they found that the toxicity of iron was low and that its presence reduced the toxicity of other metals. On the other hand, the presence of lead increased the toxicity of copper and zinc.

Interaction with sediment also influences the impact of metals. Often, over half of the trace metals are attached to sediment (MWCOG, 1983). This effectively removes the metals from the water column and reduces the availability for biological uptake and subsequent bioaccumulation (Gavin and Moore, 1982 and OWML, 1983). However, metals accumulated in bottom sediment can then be resuspended during storms (Heaney and Huber, 1978). It is important to note that the toxic effect of metals can be altered when found in conjunction with other substances. For instance, the presence of chlorides can increase the toxicity of some metals. Both metals and chlorides are common pollutants in snowpacks (see section 4.2 for more snow melt information).

4.6.3 Sources and Source Areas of Trace Metals

Research conducted in the Santa Clara Valley of California suggests that cars can be the dominant loading source for many metals of concern, such as cadmium, chromium, copper, lead, mercury and zinc (EOA, Inc., 2001). Other sources are also important and include atmospheric deposition, rooftops and runoff from industrial and residential sites.

The sources and source areas for zinc, copper, lead, chromium and cadmium are listed in Table 33. Source areas for trace metals in the urban environment include streets, parking lots, snowpacks and rooftops. Copper is often found in higher concentrations on urban streets, because some vehicles have brake pads that contain copper. For example, the Santa Clara study estimated that 50% of the total copper load was due to brake pad wear (Woodward-Clyde, 1992). Sources of lead include atmospheric deposition and diesel fuel emissions, which frequently occur along rooftops

Table 32: Percentage of In-situ Flow-through Toxicity Tests Using Daphnia *magna* and Pimephales *promelas* with Significant Toxic Effects from Lincoln Creek (Crunkilton *et al.*, 1996)

Species	Effect	Percent of Tests with Significant (p<0.05) Toxic Effects as Compared to Controls According to Exposure				
		48 hours	96 hours	7 days	14 days	17-61 days
D. *magna*	Mortality	0	N/R	36%	93%	N/R
	Reduced Reproduction	0	N/R	36%	93%	N/R
P. *promelas*	Mortality	N/R	0	0	14%	100%
	Reduced Biomass	N/R	N/R	60%	75%	N/R

N/R = Not Reported

and streets. Zinc in urban environments is a result of the wear of automobile tires (estimated 60% in the Santa Clara study), paints, and weathering of galvanized gutters and downspouts. Source area concentrations of trace metals are presented in Table 34. In general, trace metal concentrations vary considerably, but the relative rank among source areas remains relatively constant. For example, a source loading model developed for an urban watershed in Michigan estimated that parking lots, driveways and residential streets were the primary source areas for zinc, copper and cadmium loads (Steuer *et al.*, 1997).

Table 33: Metal Sources and Source Area "Hotspots" in Urban Areas

Metal	Sources	Source Area Hotspots
Zinc	tires, fuel combustion, galvanized pipes, roofs and gutters, road salts **estimate of 60% from tires*	parking lots, commercial and industrial rooftops, and streets
Copper	auto brake linings, pipes and fittings, algacides, and electroplating **estimate of 50% from brake pad wear*	parking lots, commercial roofs and streets
Lead	diesel fuel, paints and stains	parking lots, rooftops, and streets
Cadmium	component of motor oil and corrodes from alloys and plated surfaces	parking lots, rooftops, and streets
Chromium	found in exterior paints and corrodes from alloys and plated surfaces	most frequently found in industrial and commercial runoff
Sources: Bannerman et al., 1993; Barr, 1997; Steuer et al., 1997; Good, 1993; Woodward - Clyde, 1992		

Table 34: Metal Source Area Concentrations in the Urban Landscape (*F*g/l)

Source Area	Dissolved Zinc	Total Zinc	Dissolved Copper		Total Copper	Dissolved Lead		Total Lead		
Source	(1)	(2)	(1)	(2)	(2)	(1)	(3)	(1)	(3)	(2)
Commercial Parking Lot	64	178	10.7	9	15	N/R	N/R	40	N/R	22
High Traffic Street	73	508	11.2	18	46	2.1	1.7	37	25	50
Medium Traffic Street	44	339	7.3	24	56	1.5	1.9	29	46	55
Low Traffic Street	24	220	7.5	9	24	1.5	.5	21	10	33
Commercial Rooftop	263	330	17.8	6	9	20	N/R	48	N/R	9
Residential Rooftop	188	149	6.6	10	15	4.4	N/R	25	N/R	21
Residential Driveway	27	107	11.8	9	17	2.3	N/R	52	N/R	17
Residential Lawn	N/R	59	N/R	13	13	N/R	N/R	N/R	N/R	N/R
Basin Outlet	23	203	7.0	5	16	2.4	N/R	49	N/R	32
Sources: (1) Steuer et al., 1997; (2) Bannerman et al., 1993; (3) Waschbusch, 2000; N/R = Not Reported										

4.7 Hydrocarbons: PAH, Oil and Grease

Hydrocarbons are petroleum-based substances and are found frequently in urban stormwater. The term “hydrocarbons” is used to refer to measurements of oil and grease and polycyclic-aromatic hydrocarbons (PAH). Certain components of hydrocarbons, such as pyrene and benzo[b]fluoranthene, are carcinogens and may be toxic to biota (Menzie-Cura , 1995). Hydrocarbons normally travel attached to sediment or organic carbon. Like many pollutants, hydrocarbons accumulate in bottom sediments of receiving waters, such as urban lakes and estuaries. Relatively few studies have directly researched the impact of hydrocarbons on streams.

4.7.1 Concentrations

Table 35 summarizes reported EMCs of PAH and oil and grease derived from storm event monitoring at three different areas of the U.S. The limited research on oil and grease concentrations in urban runoff indicated that the highest concentrations were consistently found in commercial areas, while the lowest were found in residential areas.

4.7.2 Impacts of Hydrocarbons on Streams

The primary concern of PAH and oil and grease on streams is their potential bioaccumulation and toxicity in aquatic organisms. Bioaccumulation in crayfish, clams and fish has been reported by Masterson and Bannerman (1994); Moring and Rose (1997); and Velinsky and Cummins (1994).

Table 35: Hydrocarbon EMCs in Urban Areas

Hydrocarbon Indicator	EMC Mean	Number of Events	Source	Location
PAH (Fg/l)	3.2*	12	Menzie-Cura, 1995	MA
	7.1	19	Menzie-Cura, 1995	MA
	13.4	N/R	Crunkilton *et al.*, 1996	WI
Oil and Grease (mg/l)	1.7 **R**** 9 **C** 3 **I**	30	Baird *et al.*, 1996	TX
	3	N/R	USEPA, 1983	U.S.
	5.4*	8	Menzie-Cura, 1995	MA
	3.5	10	Menzie-Cura, 1995	MA
	3.89 **R** 13.13 **C** 7.10 **I**	N/R	Silverman *et al.*, 1988	CA
	2.35 **R** 5.63 **C** 4.86 **I**	107	Barr, 1997	MD

*N/R = Not Reported; R = Residential, C = Commercial, I = Industrial; * = geometric mean, ** = median*

Moring and Rose (1997) also showed that not all PAH compounds accumulate equally in urban streams. They detected 24 different PAH compounds in semi-permeable membrane devices (SPMDs), but only three PAH compounds were detected in freshwater clam tissue. In addition, PAH levels in the SPMDs were significantly higher than those reported in the clams.

While acute PAH toxicity has been reported at extremely high concentrations (Ireland *et al.*, 1996), delayed toxicity has also been found (Ellis, 1986). Crayfish from Lincoln Creek had a PAH concentration of 360 Fg/kg, much higher than the concentration thought to be carcinogenic (Masterson and Bannerman, 1994). By comparison, crayfish in a non-urban stream had undetectable PAH levels. Toxic effects from PAH compounds may be limited since many are attached to sediment and may be less available, with further reduction occurring through photodegradation (Ireland *et al.,* 1996).

The metabolic effect of PAH compounds on aquatic life is unclear. Crunkilton *et al.* (1996) found potential metabolic costs to organisms, but Masterson and Bannerman (1994) and MacCoy and Black (1998) did not. The long-term effect of PAH compounds in sediments of receiving waters remains a question for further study.

4.7.3 Sources and Source Areas of Hydrocarbons

In most residential stormwater runoff, hydrocarbon concentrations are generally less than 5mg/l, but the concentrations can increase to five to 10 mg/l within some commercial, industrial and highway areas (See Table 35). Specific "hotspots" for hydrocarbons include gas stations, commuter parking lots, convenience stores, residential parking areas and streets (Schueler and Shepp, 1993). These authors evaluated hydrocarbon concentrations within oil and grease separators in the Washington Metropolitan area and determined that gas stations had significantly higher concentrations of hydrocarbons and trace metals, as compared to other urban source areas. Source area research in an urban catchment in Michigan showed that commercial parking lots contributed 64% of the total hydrocarbon load (Steuer *et al.*, 1997). In addition, highways were found to be a significant contributor of hydrocarbons by Lopes and Dionne (1998).

4.8 Bacteria and Pathogens

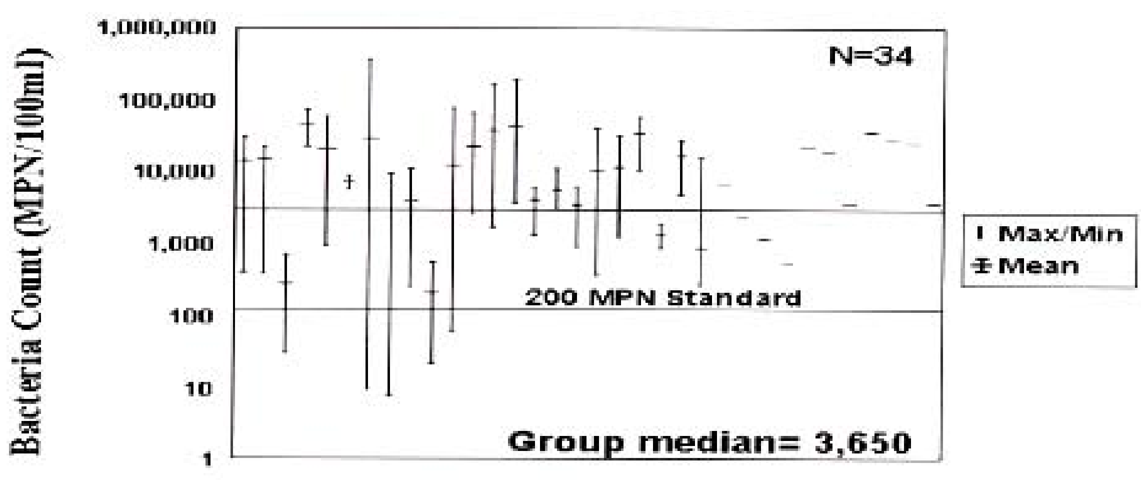
Bacteria are single celled organisms that are too small to see with the naked eye. Of particular interest are coliform bacteria, typically found within the digestive system of warm-blooded animals. The coliform family of bacteria includes fecal coliform, fecal streptococci and Escherichia coli, which are consistently found in urban stormwater runoff. Their presence confirms the existence of sewage or animal wastes in the water and indicates that other harmful bacteria, viruses or protozoans may be present, as well. Coliform bacteria are indicators of potential public health risks and not actual causes of disease.

A pathogen is a microbe that is actually known to cause disease under the right conditions. Two of the most common waterborne pathogens in the U.S. are the protozoans *Cryptosporidium parvum* and *Giardia lambia*. *Cryptosporidium* is a waterborne intestinal parasite that infects cattle and domestic animals and can be transmitted to humans, causing life-threatening problems in people with impaired immune systems (Xiao *et al.*, 2001). *Giardia* can cause intestinal problems in humans and animals when ingested (Bagley *et al.,* 1998). To infect new hosts, protozoans create hard casings known as oocysts (*Cryptosporidium*) or cysts (*Giardia*) that are shed in feces and travel through surface waters in search of a new host.

4.8.1 Concentrations

Concentrations of fecal coliform bacteria in urban stormwater typically exceed the 200 MPN/100 ml threshold set for human contact recreation (USGS, 2001b). Bacteria concentrations also tend to be highly variable from storm to storm. For example, a national summary of fecal coliform bacteria in stormwater runoff is shown in Figure 35 and Table 36. The variability in fecal coliform ranges from 10 to 500,000 MPN/100ml with a mean of 15,038 MPN/100ml (Schueler, 1999). Another national database of more than 1,600 stormwater events computed a mean concentration of 20,000

Fecal Coliform Levels in Urban Stormwater:
A National Review



Stormwater runoff levels from 34 small catchments in
13 monitoring studies conducted:
AL, AZ, ID, KY, MD, NC, NH, NY, SD, TN, TX, WA, WI

Figure 35: Fecal Coliform Levels in Urban Stormwater (Schueler, 1999)

Table 36: Bacteria EMCs in Urban Areas				
Bacteria Type	EMCs (MPN/100ml) Mean	Number of Events	Source	Location
Fecal Coliform	15,038	34	Schueler, 1999	U.S.
	20,000	1600	Pitt, 1998	U.S.
	7,653	27	Thomas and McClelland, 1995	GA
	20,000 **R*** 6900 **C** 9700 **I**	30*	Baird *et al.*, 1996	TX
	77,970	21 watersheds	Chang *et al.*, 1990	TX
	4,500	189	Varner, 1995	WA
	23,500	3	Young and Thackston, 1999	TN
Fecal Strep	35,351	17	Schueler, 1999	U.S.
	28,864 **R**	27	Thomas and McClelland, 1995	GA
	56,000 **R** * 18,000 **C** 6,100 **I**	30*	Baird *et al.*, 1996	TX

*N/R = Not Reported, R = Residential Area, C = Commercial Area, I = Industrial Area, * = Median*

MPN/100ml for fecal coliform (Pitt, 1998). Fecal streptococci concentrations for 17 urban sites across the country had a mean of 35,351 MPN/100ml (Schueler, 1999).

Young and Thackston (1999) showed that bacteria concentrations at four sites in metro Nashville were directly related to watershed IC. Increasing IC reflects the cumulative increase in potential bacteria sources in the urban landscape, such as failing septic systems, sewage overflows, dogs, and inappropriate discharges. Other studies show that concentrations of bacteria are typically higher in urban areas than rural areas (USGS, 1999a), but they are not always directly related to IC. For example, Hydroqual (1996) found that concentrations of fecal coliform in seven subwatersheds of the Kensico watershed in New York were generally higher for more developed basins, but fecal coliform concentrations did not directly increase with IC in the developed basins (Figure 36).

There is some evidence that higher concentrations of coliform are found in arid or semi-arid watersheds. Monitoring data from semi-arid regions in Austin, San Antonio, and Corpus Christi, Texas averaged 61,000, 37,500 and 40,500 MPN/100ml, respectively (Baird *et al.*,1996 and Chang *et al.* 1990). Schiff (1996), in a report of Southern California NPDES monitoring, found that median concentrations of fecal coliform in San Diego were 50,000 MPN/100ml and averaged 130,000 MPN/100ml in Los Angeles. In all of these arid and semi-arid regions, concentrations were significantly higher than the national average of 15,000 to 20,000 MPN/100ml.

Concentrations of *Cryptosporidium* and *Giardia* in urban stormwater are shown in Table 37. States *et al.* (1997) found high concentrations of *Cryptosporidium* and *Giardia* in storm samples from a combined sewer in Pittsburgh (geometric mean 2,013 oocysts/100ml and 28,881 cysts/100ml). There is evidence that urban stormwater runoff may have higher concentrations of *Cryptosporidium* and *Giardia* than other surface waters, as reported in Table 38 (Stern, 1996). Both pathogens were detected in about 50% of urban stormwater samples, suggesting some concern for drinking water supplies.

4.8.2 Impacts of Bacteria and Pathogens on Streams

Fecal coliform bacteria indicate the potential for harmful bacteria, viruses, or protozoans and are used by health authorities to determine public health risks. These standards were established to protect human health based on exposures to water during recreation and drinking. Bacteria standards for various water uses are presented in Table 39 and are all easily exceeded by typical urban stormwater concentrations. In fact, over 80,000 miles of streams and rivers are currently in non-attain-

Table 37: *Cryptosporidium* and *Giardia* EMCs

Pathogens	Units	EMCs		Number of Events	Source
		Mean	Median		
Cryptosporidium	oocysts	37.2	3.9	78	Stern, 1996
	oocysts/100ml	2013	N/R	N/R	States *et al.*, 1997
Giardia	cysts	41.0	6.4	78	Stern, 1996
	cysts/100ml	28,881	N/R	N/R	States *et al.*, 1997

N/R= Not reported

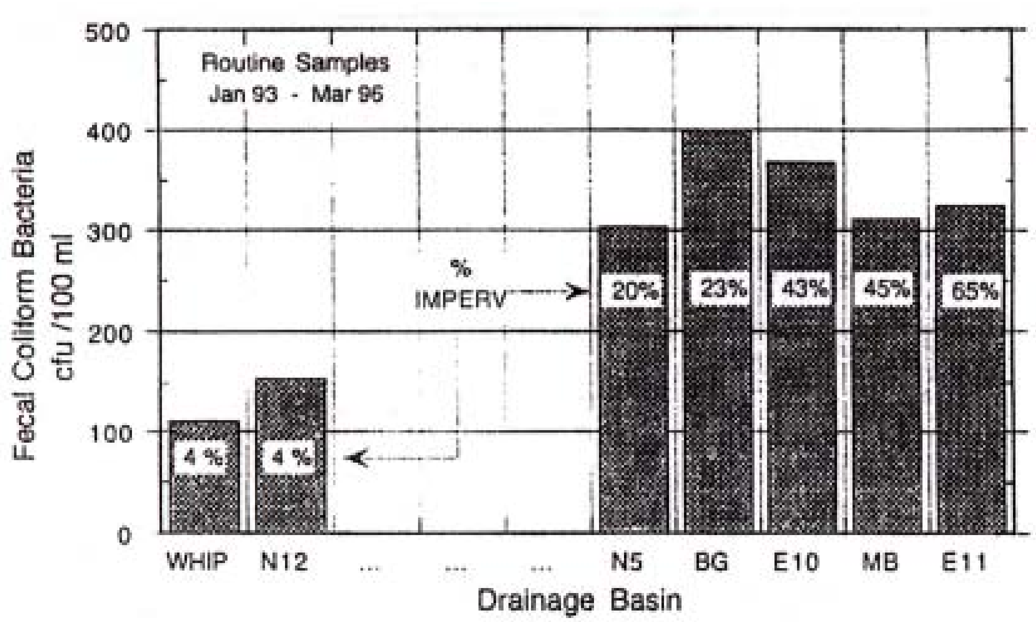


Figure 36: Relationship Between IC and Fecal Coliform Concentrations in New York Streams (Hydroqual, 1996)

Table 38: Percent Detection of *Giardia* cysts and *Cryptosporidium* oocysts in Subwatersheds and Wastewater Treatment Plant Effluent in the New York City Water Supply Watersheds (Stern, 1996)

Source Water Sampled	Number of Sources/ Number of Samples	Percent Detection			
		Total *Giardia*	Confirmed *Giardia*	Total *Cryptosporidium*	Confirmed *Cryptosporidium*
Wastewater Effluent	8/147	41.5%	12.9%	15.7%	5.4%
Urban Subwatershed	5/78	41.0%	6.4%	37.2%	3.9%
Agricultural Subwatershed	5/56	30.4%	3.6%	32.1%	3.6%
Undisturbed Subwatershed	5/73	26.0%	0.0%	9.6%	1.4%

Table 39: Typical Coliform Standards for Different Water Uses (USEPA, 1998)

Water Use	Microbial Indicator	Typical Water Standard
Water Contact Recreation	Fecal Coliform	<200 MPN per 100ml
Drinking Water Supply	Fecal Coliform	<20 MPN per 100ml
Shellfish Harvesting	Fecal Coliform	<14 MPN/ 100ml
Treated Drinking Water	Total Coliform	No more than 1% coliform positive samples per month
Freshwater Swimming	E.Coli	<126 MPN per 100ml

Important Note: Individual state standards may employ different sampling methods, indicators, averaging periods, averaging methods, instantaneous maximums and seasonal limits. MPN = most probable number. Higher or lower limits may be prescribed for different water use classes.

ment status because of high fecal coliform levels (USEPA, 1998).

4.8.3 Sources and Source Areas of Bacteria and Pathogens

Sources of coliform bacteria include waste from humans and wildlife, including livestock and pets. Essentially, any warm-blooded species that is present in significant numbers in a watershed is a potential culprit. Source identification studies, using methods such as DNA fingerprinting, have put the blame on species such as rats in urban areas, ducks and geese in stormwater ponds, livestock from hobby farms, dogs and even raccoons (Blankenship, 1996; Lim and Olivieri, 1982; Pitt, 1998; Samadpour and Checkowitz, 1998).

Transport of bacteria takes place through direct surface runoff, direct inputs to receiving waters, or indirect secondary sources. Source areas in the urban environment for direct runoff include lawns and turf, driveways, parking lots and streets. For example, dogs have high concentrations of fecal coliform in their feces and have a tendency to defecate in close proximity to IC (Schueler, 1999). Weiskel *et al. (*1996) found that direct inputs of fecal coliform from waterfowl can be very important; these inputs accounted for as much as 67% of the annual coliform load to Buttermilk Bay, Massachusetts.

Indirect sources of bacteria include leaking septic systems, illicit discharges, sanitary sewer overflows (SSOs), and combined sewer overflows (CSOs). These sources have the potential to deliver high coliform concentrations to urban streams. In fact, extremely high bacteria concentrations are usually associated with wastewater discharges. CSOs and SSOs occur when the flow into the sewer exceeds the capacity of the sewer lines to drain them. CSOs result from stormwater flow in the lines, and SSOs are a result of infiltration problems or blockages in the lines.

Illicit connections from businesses and homes to the storm drainage system can discharge sewage or washwater into receiving waters. Illicit discharges can often be identified by baseflow sampling of storm sewer systems. Leaking septic systems are estimated to comprise between 10 and 40% of the systems, and individual inspections are the best way to determine failing systems (Schueler, 1999).

There is also evidence that coliform bacteria can survive and reproduce in stream sediments and storm sewers (Schueler, 1999). During a storm event, they often become resuspended and add to the in-stream bacteria load. Source area studies reported that end of pipe concentrations were an order of magnitude higher than any source area on the land surface; therefore, it is likely that the storm sewer system itself acts as a source of fecal coliform (Bannerman *et al.*, 1993 and Steuer *et al.*, 1997). Resuspension of fecal coliform from fine stream sediments during storm events has been reported in New Mexico (NMSWQB, 1999). The sediments in-stream and in the storm sewer system may be significant contributors to the fecal coliform load.

Sources of *Cryptosporidium* and *Giardia* include human sewage and animal feces. *Cryptosporidium* is commonly found in cattle, dogs and geese. Graczyk *et al.* (1998) found that migrating Canada geese were a vector for *Cryptosporidium* and *Giardia*, which has implications for water quality in urban ponds that support large populations of geese.

4.9 Organic Carbon

Total organic carbon (TOC) is often used as an indicator of the amount of organic matter in a water sample. Typically, the more organic matter present in water, the more oxygen consumed, since oxygen is used by bacteria in the decomposition process. Adequate levels of dissolved oxygen in streams and receiving waters are important because they are critical to maintain aquatic life. Organic carbon is routinely found in urban stormwater, and high concentrations can result in an increase in Biochemical Oxygen Demand (BOD) and Chemical Oxygen Demand (COD). BOD and COD are measures of the oxygen demand caused by the decay of organic matter.

4.9.1 Concentrations

Urban stormwater has a significant ability to exert a high oxygen demand on a stream or receiving water, even two to three weeks after an individual storm event (Field and Pitt, 1990). Average concentrations of TOC, BOD and COD in urban stormwater are presented in Table 40. Mean concentrations of TOC, BOD and COD during storm events in nationwide studies were 17 mg/l, 14.1 mg/l and 52.8 mg/l, respectively (Kitchell, 2001 and Smullen and Cave,1998).

4.9.2 Impacts of Organic Carbon on Streams

TOC is primarily a concern for aquatic life because of its link to oxygen demand in streams, rivers, lakes and estuaries. The initial effect of increased concentrations of TOC, BOD or COD in stormwater runoff may be a depression in oxygen levels, which may persist for many days after a storm, as deposited organic matter gradually decomposes (Field and Pitt, 1990).

TOC is also a concern for drinking water quality. Organic carbon reacts with chlorine during the drinking water disinfection process and forms trihalomethanes and other disinfection by-products, which can be a serious drinking water quality problem (Water, 1999). TOC concentrations greater than 2 mg/l in treated water and 4 mg/l in source water can result in unacceptably high levels of disinfection byproducts and must be treated to reduce TOC or remove the disinfection byproducts (USEPA, 1998). TOC can also be a carrier for other pollutants, such as trace metals, hydrocarbons and nutrients.

4.9.3 Sources and Source Areas of Total Organic Carbon

The primary sources of TOC in urban areas appear to be decaying leaves and other organic matter, sediment and combustion by-products. Source areas include curbs, storm drains, streets and stream channels. Dartiguenave *et al.* (1997) determined that about half of the annual TOC load in urban watersheds of Austin, TX was derived from the eroding streambanks.

Table 40: EMCs for Organic Carbon in Urban Areas

Organic Carbon Source	EMCs (mg/l)		Number of Events	Source
	Mean	Median		
Total Organic Carbon (TOC)	32.0	N/R	423	Barrett and Malina, 1998
	17	15.2	19 studies	Kitchell, 2001
Biological Oxygen Demand (BOD)	14.1	11.5	1035	Smullen and Cave, 1998
	10.4	8.4	474	USEPA, 1983
Chemical Oxygen Demand (COD)	52.8	44.7	2639	Smullen and Cave, 1998
	66.1	55	1538	USEPA, 1983

N/R = Not Reported

4.10 MTBE

Methyl tertiary butyl-ether (MTBE) is a volatile organic compound (VOC) that is added to gasoline to increase oxygen levels, which helps gas burn cleaner (called an oxygenate). MTBE has been used as a performance fuel additive since the 1970s. In 1990, the use of oxygenates was mandated by federal law and concentrations of MTBE in gasoline increased. Today, MTBE is primarily used in large metropolitan areas that experience air pollution problems. Since 1990, MTBE has been detected at increasing levels in both surface water and groundwater and is one of the most frequently detected VOCs in urban watersheds (USGS, 2001a). EPA has declared MTBE to be a potential human carcinogen at high doses. In March 2000, a decision was made by EPA to follow California's lead to significantly reduce or eliminate the use of MTBE in gasoline.

4.10.1 Concentrations

MTBE is highly soluble in water and therefore not easily removed once it enters surface or ground water. Delzer (1999) detected the presence of MTBE in 27% of the shallow wells monitored in eight urban areas across the country (Figure 37). Detection frequency was significantly higher in New England and Denver, as shown in Table 41. In a second study conducted in 16 metropolitan areas, Delzer (1999) found that 83% of MTBE detections occurred between October and March, the time when MTBE is primarily used as a fuel additive. The median MTBE concentration was 1.5 ppb, well below EPA's draft advisory level of 20 ppb (Delzer, 1996).

4.10.2 Impacts of MTBE on Streams

The primary concerns regarding MTBE are that it is a known carcinogen to small mammals, a suspected human carcinogen at higher

Table 41: MTBE Detection Frequency

Location	Detection Frequency	Source	Year
211 shallow wells in eight urban areas	27%	Delzer	1999
Surface water samples in 16 metro areas	7%	Delzer	1996

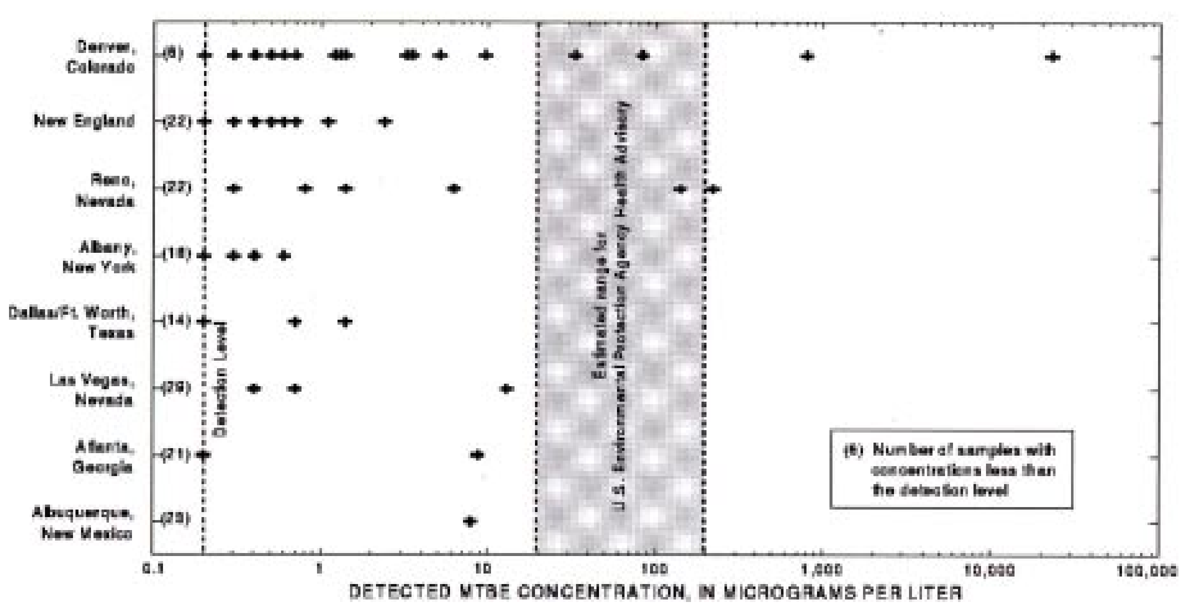


Figure 37: MTBE Concentrations in Surface Water from Eight Cities (Delzer, 1996)

doses and may possibly be toxic to aquatic life in small streams (Delzer, 1996). MTBE can also cause taste and odor problems in drinking water at fairly low concentrations. EPA issued a Drinking Water Advisory in 1997 that indicated that MTBE concentrations less than 20 ppb should not cause taste and odor problems for drinking water. However, the Association of California Water Agencies reports that some consumers can detect MTBE at levels as low as 2.5 ppb (ACWA, 2000). Because MTBE is frequently found in groundwater wells, it is thought to be a potential threat to drinking water (Delzer, 1999). For example, Santa Monica, California reportedly lost half of its groundwater drinking water supply due to MTBE contamination (Bay and Brown, 2000). MTBE has also been detected in human blood, especially in people frequently exposed to gasoline, such as gas station attendants (Squillace *et al.*, 1995).

4.10.3 Sources and Source Areas of MTBE

Since MTBE is a gasoline additive, its potential sources include any area that produces, transports, stores, or dispenses gasoline, particularly areas that are vulnerable to leaks and spills. Leaking underground storage tanks are usually associated with the highest MTBE concentrations in groundwater wells (Delzer, 1999). Vehicle emissions are also an important source of MTBE. Elevated levels are frequently observed along road corridors and drainage ditches. Once emitted, MTBE can travel in stormwater runoff or groundwater. Main source areas include heavily used multi-lane highways. Gas stations may also be a hotspot source area for MTBE contamination.

Another potential source of MTBE is watercraft, since two cycle engines can discharge as much as 20 to 30% of their fuel through the exhaust (Boughton and Lico, 1998). MTBE concentrations are clearly associated with increased use of gas engines, and there is concern that MTBE is an increasing component of atmospheric deposition (Boughton and Lico, 1998 and UC Davis, 1998).

4.11 Pesticides

Pesticides are used in the urban environment to control weeds, insects and other organisms that are considered pests. EPA estimates that nearly 70 million pounds of active pesticide ingredients are applied to urban lawns each year as herbicides or insecticides. Herbicides are used on urban lawns to target annual and perennial broadleaf weeds, while insecticides are used to control insects. Many types of pesticides are available for use in urban areas. Immerman and Drummond (1985) report that 338 different active ingredients are applied to lawns and gardens nationally. Each pesticide varies in mobility, persistence and potential aquatic impact. At high levels, many pesticides have been found to have adverse effects on ecological and human health. Several recent research studies by the USGS have shown that insecticides are detected with the greatest frequency in urban streams, and that pesticide detection frequency increases in proportion to the percentage of urban land in a watershed (Ferrari *et al.*, 1997; USGS, 1998, 1999a-b, 2001b). A national assessment by the USGS

Table 42: Median Concentrations and Detection Frequency of Herbicides and Insecticides in Urban Streams

Pollutant	Detection Frequency	Median Concentration (Fg/l)	Number of Samples	Source
Insecticides				
Diazinon	75%	0.025	326	USGS, 1998b
	92%	0.55	76	Brush *et al.*, 1995
	17%	0.002	1795	Ferrari *et al.*, 1997
Chlorpyrifos	41%	Non Detect	327	USGS, 1998b
	14%	0.004	1218	Brush *et al.*, 1995
Carbaryl	46%	Non Detect	327	USGS, 1998b
	22%	0.003	1128	Ferrari *et al.*, 1997
Herbicides				
Atrazine	86%	0.023	327	USGS, 1998b
	72%	0.099	2076	Ferrari *et al.*, 1997
Prometon	84%	0.031	327	USGS, 1998b
	56%	0.029	1531	Ferrari *et al.*, 1997
Simazine	88%	0.039	327	USGS, 1998b
	17%	0.046	1995	Ferrari *et al.*, 1997
2,4 -D	67%	1.1	11	Dindorf, 1992
	17%	0.035	786	Ferrari *et al.*, 1997
Dicamba	22%	1.8	4	Dindorf, 1992
MCPP	56%	1.8	10	Dindorf, 1992
MCPA	28%	1.0	5	Dindorf, 1992

(2001a) also indicates that insecticides are usually detected at higher concentrations in urban streams than in agricultural streams.

4.11.1 Concentrations

Median concentrations and detection frequency for common pesticides are shown in Table 42. Herbicides that are frequently detected in urban streams include atrazine; simazine; prometon; 2,4-D; dicamba; MCPP; and MCPA. Insecticides are also frequently encountered in urban streams, including diazinon, chlorpyrifos, malathion, and carbaryl. A USGS (1996) study monitored 16 sites in Gills Creek in Columbia, South Carolina over four days. This study reported that pesticide detection frequency increased as percent urban land increased.

Wotzka *et al.* (1994) monitored herbicide levels in an urban stream in Minneapolis, Minnesota during more than 40 storms. They found herbicides, such as 2,4-D; dicamba; MCPP; and MCPA in 85% of storm runoff events sampled. Total herbicide EMCs ranged from less than one to 70 µg/l. Ferrari *et al.* (1997) analyzed 463 streams in the mid-Atlantic region for the presence of 127 pesticide compounds. At least one pesticide was detected at more than 90% of the streams sampled.

Diazinon is one of the most commonly detected insecticides in urban stormwater runoff and dry weather flow. Diazinon was detected in 75% of National Water Quality Assessment (NAWQA) samples, 92% of stormflow samples from Texas, and 100% of urban stormflow samples in King County, Washington (Brush *et al.*, 1995 and USGS, 1999b). Diazinon is most frequently measured at concentrations greater than freshwater aquatic life criteria in urban stormwater (USGS, 1999a). USGS reports that diazinon concentrations were generally higher during urban stormflow (Ferrari *et al.,* 1997).

4.11.2 Impacts of Pesticides on Streams

Many pesticides are known or suspected carcinogens and can be toxic to humans and aquatic species. However, many of the known health effects require exposure to higher concentrations than typically found in the environment, while the health effects of chronic exposure to low levels are generally unknown (Ferrari *et al.*, 1997).

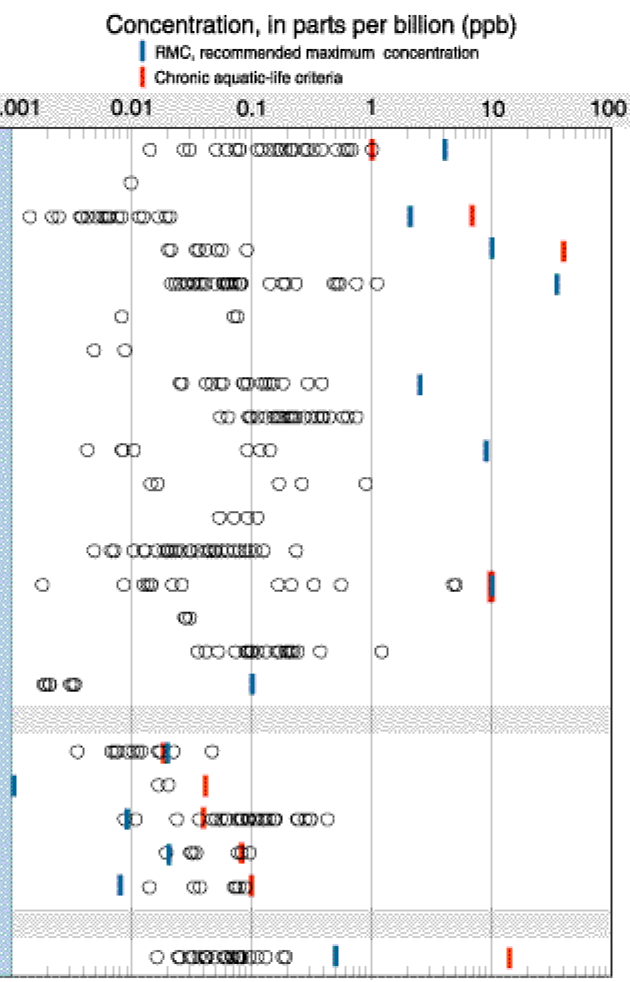
Studies that document the toxicity of insecticides and herbicides in urban stormwater have been focused largely on diazinon. Diazinon is responsible for the majority of acute toxicity in stormwater in Alameda County, California and King County, Washington (S.R. Hansen & Associates, 1995). Concentrations of diazinon in King County stormwater frequently exceed the freshwater aquatic life criteria (Figure 38). Similarly, research on Sacramento, California streams revealed acute toxicity for diazinon in 100% of stormwater samples using *Ceriodaphnia* as the test organism (Connor, 1995). Diazinon has a half-life of 42 days and is very soluble in water, which may explain its detection frequency and persistence in urban stormwater. Diazinon is also reported to attach fairly readily to organic carbon; consequently, it is likely re-suspended during storm events.

Insecticide concentrations exceeding acute and chronic toxicity thresholds for test organisms such as *Ceriodaphnia* have frequently been found in urban stormwater in New York, Texas, California, and Washington (Scanlin and Feng, 1997; Brush *et al.*, 1995; USGS, 1999b). The possibility exists that pesticides could have impacts on larger bodies of water, but there is a paucity of data on the subject at this time.

4.11.3 Sources and Source Areas of Pesticides

Sources for pesticides in urban areas include applications by homeowners, landscaping contractors and road maintenance crews. Source areas for pesticides in urban areas include lawns in residential areas; managed turf, such as golf courses, parks, and ball fields; and rights-of-way in nonresidential areas. Storage areas, which are subject to spills and leaks, can also be a source area. A study in San Francisco was able to trace high diazinon concentrations in some streams back to just a few households which had applied the pesticide at high levels (Scanlin and Feng, 1997). Two herbicides, simazine and atrazine, were detected in over 60% of samples in King County, WA stormwater but were not identified as being sold in retail stores. It is likely these herbicides are applied to nonresidential areas such as rights-of-way, parks and recreational areas (USGS, 1999b). Because pesticides are typically applied to turf, IC is not a direct indicator for pesticide concentrations, although they can drift onto paved surfaces and end up in stormwater runoff.

Active ingredient and trade name[1] example in parentheses	Freshwater aquatic-life criteria, in parts per billion (-- no aquatic-life criteria exist)		
Herbicide	Acute	Chronic	RMC
2,4-D (Weedone)	[4]10	[4]1	[5,2]4
Acetochlor (Guardian)	--	--	--
Atrazine (AAtrex)	[4]70	[4]7	[5]2
Dicamba (Banvel)	[4]390	[4]39	[5]10
Dichlobenil (Casoron)	--	--	[2]37
Dichlorprop (2,4-DP)	--	--	--
EPTC (Eptam)	--	--	--
MCPA (Kilsem)	--	--	[5]2.6
MCPP (Mecoprop)	--	--	--
Metolachlor (Dual)	--	--	[5]8
Napropamide (Devrinol)	--	--	--
Oxadiazon (Ronstar)	--	--	--
Prometon (Pramitol)	--	--	--
Simazine (Princep)	[4]100	[4]10	[2,5]10
Tebuthiuron (Spike)	--	--	--
Triclopyr (Garlon)	[4]5,600	[4]560	--
Trifluralin (Treflan)	--	--	[5]0.1
Insecticide			
Carbaryl (Sevin)	[4]0.17	[4]0.017	[2]0.02
Chlorpyrifos (Dursban)	[3,7]0.083	[3,7]0.041	[2]0.001
Diazinon (Diazinon)	[6]0.16	[6]0.04	[2]0.009
gamma-HCH (Lindane)	[3]0.95	[7]0.08	[2]0.02
Malathion (Malathion)	--	[3]0.1	[2]0.008
Fungicide			
Pentachlorophenol (Penta)	[3,8]19	[3,8]15	[5]0.5



[1] Use of trade names is for descriptive purposes and does not imply endorsement by the U.S. Geological Survey, the Department of Ecology, or King County
[2] National Academy of Sciences and National Academy of Engineering, 1973
[3] U.S. Environmental Protection Agency, 1998
[4] Norris and Dost, 1991, Table 3
[5] Ministers of Health Canada and Environment Canada, 1995
[6] Menconi and Cox, 1994
[7] State of Washington, 1992
[8] Criteria are pH dependent; values shown calculated at pH of 7.8

Figure 38: Concentrations of Pesticides in Stormwater in King County, WA (S.R. Hansen & Associates, 1995 and USGS, 1999b)

4.12 Deicers

Deicers are substances used to melt snow and ice to keep roads and walking areas safe. The most commonly used deicer is sodium chloride, although it may also be blended with calcium chloride or magnesium chloride. Other less frequently used deicers include urea and glycol, which are primarily used at airports to deice planes. Table 43 summarizes the composition, use and water quality effects of common deicers.

Chlorides are frequently found in snowmelt and stormwater runoff in most regions that experience snow and ice in the winter months (Oberts, 1994 and Sherman, 1998). Figure 39 shows that the application of deicer salts has increased since 1940 from 200,000 tons to 10 to 20 million tons per year in recent years (Salt Institute, 2001). Several U.S. and Canadian studies indicate severe inputs of road salts on water quality and aquatic life (Environment Canada, 2001 and Novotny *et al.*, 1999).

Table 43: Use and Water Quality Effect of Snowmelt Deicers (Ohrel, 1995; Sills and Blakeslee, 1992)

Deicer	Description	Use	Water Quality Effect
Chlorides	Chloride based deicer usually combined with Na, Ca or Mg	Road Deicer and Residential Use	Cl complexes can release heavy metals, affect soil permeability, impacts to drinking water, potential toxic effects to small streams
Urea	Nitrogen-based fertilizer product	Used as alternative to glycol	Increased nitrogen in water and potential toxicity to organisms
Ethylene Glycol	Petroleum based organic compounds, similar to antifreeze	Used at airports for deicing planes	Toxicity effects, high BOD and COD, hazardous air pollutant

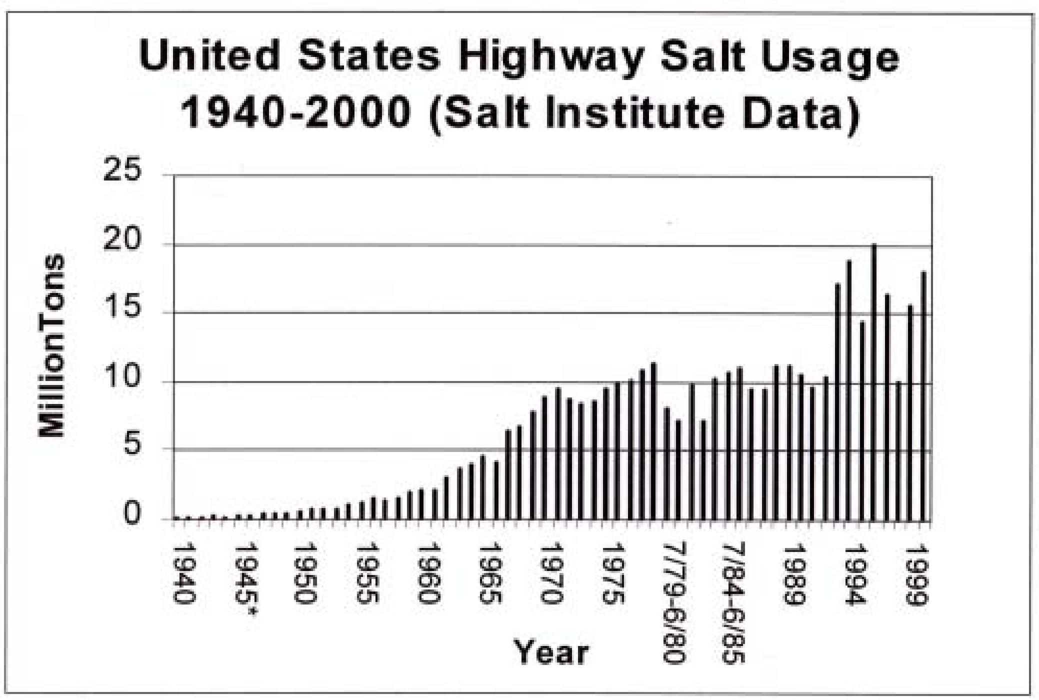


Figure 39: U.S. Highway Salt Usage Data (Salt Institute, 2001)

4.12.1 Concentrations

Chloride concentrations in snowmelt runoff depend on the amount applied and the dilution in the receiving waters. Data for snowmelt and stormwater runoff from several studies are presented in Table 44. For example, chloride concentrations in Lincoln Creek in Wisconsin were 1,612 mg/l in winter snowmelt runoff, as compared to 40 mg/l in non-winter runoff (Novotny *et al.*, 1999 and Masterson and Bannerman, 1994). Chloride concentrations in the range of 2,000 to 5,000 mg/l have been reported for Canadian streams (Environment Canada, 2001). Novotny *et al.* (1999) monitored chloride concentrations in snowmelt near Syracuse, New York and found that residential watersheds had higher chloride concentrations than rural watersheds.

Concentrations of glycol in stormwater runoff are also highly variable and depend on the amount of deicer used, the presence of a recovery system, and the nature of the precipitation event. Corsi *et al*. (2001) monitored streams receiving stormwater runoff from a Wisconsin airport. They found concentrations of propylene glycol as high as 39,000 mg/l at airport outfall sites during deicing operations and concentrations of up to 960 mg/l during low-flow sampling at an airport outfall site.

4.12.2 Impacts of Deicers on Streams

Chloride levels can harm aquatic and terrestrial life and contaminate groundwater and drinking water supplies (Ohrel, 1995). Generally, chloride becomes toxic to many organisms when it reaches concentrations of 500 to1,000 mg/l (Environment Canada, 2001). These concentrations are common in small streams in snow regions, at least for short periods of time. Many plant species are relatively intolerant to high salt levels in wetland swales and roadside corridors. Fish are also negatively affected by high chloride concentrations, with sensitivity as low as 600 mg/l for some species (Scott and Wylie, 1980).

Table 45 compares the maximum chloride concentrations for various water uses in eight states (USEPA, 1988). Snowmelt chloride concentrations typically exceed these levels.

Table 44: EMCs for Chloride in Snowmelt and Stormwater Runoff in Urban Areas

Form of Runoff	EMCs (mg/l) Mean	Number of Events	Sources	Location
Snowmelt	116*	49	Oberts, 1994	MN
	2119	N/R	Sherman, 1998	Ontario
	1267 **R** 474 **U**	N/R	Novotny *et al.*, 1999	NY
	1612	N/R	Masterson and Bannerman, 1994	WI
	397	282	Environment Canada, 2001	Ontario, Canada
Non-winter Storm Event	42	61	Brush *et al.*, 1995	TX
	45	N/R	Sherman, 1998	Ontario
	40.5	N/R	Masterson and Bannerman, 1994	WI

*N/R = Not Reported, R = residential, U = urban, * = Median*

Chloride is a concern in surface drinking water systems because it can interfere with some of the treatment processes and can cause taste problems at concentrations as low as 250 mg/l. Chloride is also extremely difficult to remove once it enters the water.

Glycol-based deicers have been shown to be highly toxic at relatively low concentrations in streams receiving airport runoff. These deicers contain many proprietary agents, which may increase their toxicity and also make it very difficult to set standards for their use (Hartwell *et al.*, 1995). Corsi *et al.* (2001) observed acute toxicity of *Ceriodaphnia dubia*, *Pimephelas promelax*, *Hyalela azteca*, and *Chironimus tentans* in Wisconsin streams that experienced propylene glycol concentrations of 5,000 mg/l or more. Chronic toxicity was observed for *Ceriodaphnia dubia* and *Pimephelas promelax* at propylene glycol concentrations of 1,500 mg/l in the same study. In addition, glycol exerts an extremely high BOD on receiving waters, which can quickly reduce or eliminate dissolved oxygen. Glycol can also be toxic to small animals that are attracted by its sweet taste (Novotny *et al.*, 1999).

As with many urban pollutants, the effects of chloride can be diluted in larger waterbodies. In general, small streams are more likely to experience chloride effects, compared to rivers, which have a greater dilution ability.

4.12.3 Sources and Source Areas of Deicers

The main sources for deicers in urban watersheds include highway maintenance crews, airport deicing operations, and homeowner applications. Direct road application is the largest source of chloride, by far. Source areas include roads, parking lots, sidewalks, storm drains, airport runways, and snow collection areas. Because deicers are applied to paved surfaces, the primary means of transport to streams is through stormwater and meltwater runoff. Therefore, concentrations of deicer compounds are typically associated with factors such as road density or traffic patterns.

Table 45: Summary of State Standards for Salinity of Receiving Waters (USEPA, 1988)

State	Limiting Concentration (mg/l)	Beneficial Use
CO	250*	Drinking water
IL	500	General water supply
	250	Drinking water
IN	500	Drinking water
MA	250	Class A waters
MN	250	Drinking water
	500	Class A fishing and recreation
OH	250	Drinking water
SD	250	Drinking water
	100	Fish propagation
VA	250	Drinking water

** Monthly average*

4.13 Conclusion

IC collects and accumulates pollutants deposited from the atmosphere, leaked from vehicles, or derived from other sources. The pollutants build up over time but are washed off quickly during storms and are often efficiently delivered to downstream waters. This can create water quality problems for downstream rivers, lakes and estuaries.

As a result of local and national monitoring efforts, we now have a much better understanding of the nature and impacts of stormwater pollution. The typical sample of urban stormwater is characterized by high levels of many common pollutants such as sediment, nutrients, metals, organic carbon, hydrocarbons, pesticides, and fecal coliform bacteria. Other pollutants that have more recently become a concern in urban areas include MTBE, deicers, and the pathogens *Cryptosporidium* and *Giardia*. Concentrations of most stormwater pollutants can be characterized, over the long run, by event mean storm concentrations. Monitoring techniques have also allowed researchers to identify source areas for pollutants in the urban environment, including stormwater hotspots, which generate higher pollutant loads than normal development.

In general, most monitoring data shows that mean pollutant storm concentrations are higher in urban watersheds than in non-urban ones. For many urban pollutants, EMCs can be used to predict stormwater pollutant loads for urban watersheds, using IC as the key predictive variable. While a direct relationship between IC and pollutant concentrations does not usually exist, IC directly influences the volume of stormwater and hence, the total load. A few exceptions are worth noting. MTBE, deicers, and PAH appear to be related more to traffic or road density than IC. Additionally, MTBE and PAH concentrations may be greater at hotspot source areas, which are not always widely or uniformly distributed across a watershed. Pesticides, bacteria and pathogens are often associated with turf areas rather than IC. Bacteria and pathogen sources also include direct inputs from wildlife and inappropriate sewage discharges that are not uniformly distributed across a watershed and are not directly related to IC.

Further research into the relationship between stormwater pollutant loads and other watershed indicators may be helpful. For example, it would be interesting to see if turf cover is a good indicator of stream quality for impacted streams. Other important watershed indicators worth studying are the influence of watershed treatment practices, such as stormwater practices and stream buffers.

The direct effects of stormwater pollutants on aquatic systems appears to be a function of the size of the receiving water and the initial health of the aquatic community. For example, a small urban stream receiving high stormwater pollutant concentrations would be more likely to experience impacts than a large river, which is diluted by other land uses. Likewise, organisms in sensitive streams should be more susceptible to stormwater pollutants than pollution-tolerant organisms found in non-supporting streams.

Overall, the following conclusions can be made:

- Sediment, nutrient and trace metal loads in stormwater runoff can be predicted as a function of IC, although concentrations are not tightly correlated with watershed IC.
- Violations of bacteria standards are indirectly associated with watershed IC.
- It is not clear whether loads of hydrocarbons, pesticides or chlorides can be predicted on the basis of IC at the small watershed level.
- More research needs to be conducted to evaluate the usefulness of other watershed indicators to predict stormwater pollutant loads. For example, traffic, road density or hotspots may be useful in predicting MTBE, deicer and hydrocarbon loads. Also, watershed turf cover may be useful in predicting pesticide and bacterial loads.

- Most research on pollutants in stormwater runoff has been conducted at the small watershed level. Additional research is needed to evaluate the impact of watershed treatment, such as stormwater and buffer practices to determine the degree to which these may change stormwater concentrations or loads.
- Regional differences are evident for many stormwater pollutants, and these appear to be caused by either differences in rainfall frequency or snowmelt.

Chapter 5: Biological Impacts of Impervious Cover

This chapter reviews research on the impact of urbanization on the aquatic community, focusing on aquatic insects, fish, amphibians, freshwater mussels, and freshwater wetlands. Specifically, the relationship between the health of the aquatic community and the amount of watershed IC is analyzed within the context of the Impervious Cover Model (ICM).

The chapter is organized as follows:

5.1 Introduction
5.2 Indicators and General Trends
5.3 Effects on Aquatic Insect[1] Diversity
5.4 Effects on Fish Diversity
5.5 Effects on Amphibian Diversity
5.6 Effects on Wetland Diversity
5.7 Effects on Freshwater Mussel Diversity
5.8 Conclusion

5.1 Introduction

A number of studies, crossing different ecoregions and utilizing various techniques, have examined the link between watershed urbanization and its impact on stream and wetland biodiversity. These studies reveal that a relatively small amount of urbanization has a negative effect on aquatic diversity, and that as watersheds become highly urban, aquatic diversity becomes extremely degraded. As documented in prior chapters, hydrologic, physical, and water quality changes caused by watershed urbanization all stress the aquatic community and collectively diminish the quality and quantity of available habitat. As a result, these stressors generally cause a decline in biological diversity, a change in trophic structure, and a shift towards more pollution-tolerant organisms.

Many different habitat conditions are critical for supporting diverse aquatic ecosystems. For example, streambed substrates are vulnerable to deposition of fine sediments, which affects spawning, egg incubation and fry-rearing. Many aquatic insect species shelter in the large pore spaces among cobbles and boulders, particularly within riffles. When fine sediment fills these pore spaces, it reduces the quality and quantity of available habitat. The aquatic insect community is typically the base of the food chain in streams, helps break down organic matter and serves as a food source for juvenile fish.

Large woody debris (LWD) plays a critical role in the habitat of many aquatic insects and fish. For example, Bisson *et al.* (1988) contend that no other structural component is more important to salmon habitat than LWD, especially in the case of juvenile coho salmon. Loss of LWD due to the removal of stream side vegetation can significantly hinder the survival of more sensitive aquatic species. Since LWD creates different habitat types, its quality and quantity have been linked to salmonid rearing habitat and the ability of multiple fish species to coexist in streams.

The number of stream crossings (e.g., roads, sewers and pipelines) has been reported to increase directly in proportion to IC (May *et al*., 1997). Such crossings can become partial or total barriers to upstream fish migration, particularly if the stream bed downcuts below the fixed elevation of a culvert or pipeline. Fish barriers can prevent migration and recolonization of aquatic life in many urban streams.

Urbanization can also increase pollutant levels and stream temperatures. In particular, trace metals and pesticides often bind to sediment particles and may enter the food chain, particularly by aquatic insects that collect and filter particles. While in-stream data is rare, some data are available for ponds. A study of trace

[1] *Throughout this chapter, the term "aquatic insects" is used rather than the more cumbersome but technically correct "benthic macroinvertebrates."*

metal bioaccumulation of three fish species found in central Florida stormwater ponds discovered that trace metal levels were significantly higher in urban ponds than in non-urban control ponds, often by a factor of five to 10 (Campbell, 1995; see also Karouna-Renier, 1995). Although typical stormwater pollutants are rarely acutely toxic to fish, the cumulative effects of sublethal pollutant exposure may influence the stream community (Chapter 4).

Table 46 summarizes some of the numerous changes to streams caused by urbanization that have the potential to alter aquatic biodiversity. For a comprehensive review of the impacts of urbanization on stream habitat and biodiversity, the reader should consult Wood and Armitage (1997) and Hart and Finelli (1999).

Table 46: Review of Stressors to Urban Streams and Effects on Aquatic Life

Stream Change	Effects on Organisms
Increased flow volumes/ Channel forming storms	Alterations in habitat complexity Changes in availability of food organisms, related to timing of emergence and recovery after disturbance Reduced prey diversity Scour-related mortality Long-term depletion of LWD Accelerated streambank erosion
Decreased base flows	Crowding and increased competition for foraging sites Increased vulnerability to predation Increased fine sediment deposition
Increase in sediment transport	Reduced survival of eggs and alevins, loss of habitat due to deposition Siltation of pool areas, reduced macroinvertebrate reproduction
Loss of pools and riffles	Shift in the balance of species due to habitat change Loss of deep water cover and feeding areas
Changes in substrate composition	Reduced survival of eggs Loss of inter-gravel fry refugial spaces Reduced aquatic insect production
Loss of LWD	Loss of cover from predators and high flows Reduced sediment and organic matter storage Reduced pool formation and organic substrate for aquatic insects
Increase in temperature	Changes in migration patterns Increased metabolic activity, increased disease and parasite susceptibility Increased mortality of sensitive fish
Creation of fish blockages	Loss of spawning habitat for adults Inability to reach overwintering sites Loss of summer rearing habitat, Increased vulnerability to predation
Loss of vegetative rooting systems	Decreased channel stability Loss of undercut banks Reduced streambank integrity
Channel straightening or hardening	Increased stream scour Loss of habitat complexity
Reduction in water quality	Reduced survival of eggs and alevins Acute and chronic toxicity to juveniles and adult fish Increased physiological stress
Increase in turbidity	Reduced survival of eggs Reduced plant productivity Physiological stress on aquatic organisms
Algae blooms	Oxygen depletion due to algal blooms, increased eutrophication rate of standing waters

5.2 Indicators and General Trends

Stream indicators are used to gauge aquatic health in particular watersheds. The two main categories of stream indicators are **biotic** and **development** indices. **Biotic** indices use stream diversity as the benchmark for aquatic health and use measures, such as species abundance, taxa richness, EPT Index, native species, presence of pollution-tolerant species, dominance, functional feeding group comparisons, or proportion with disease or anomalies. **Development** indices evaluate the relationship between the degree of watershed urbanization and scores for the biotic indices. Common development indices include watershed IC, housing density, population density, and percent urban land use.

5.2.1 Biological Indicators

Biotic indices are frequently used to measure the health of the aquatic insect or fish community in urban streams. Because many aquatic insects have limited migration patterns or a sessile mode of life, they are particularly well-suited to assess stream impacts over time. Aquatic insects integrate the effects of short-term environmental variations, as most species have a complex but short life cycle of a year or less. Sensitive life stages respond quickly to environmental stressors, but the overall community responds more slowly. Aquatic insect communities are comprised of a broad range of species, trophic levels and pollution tolerances, thus providing strong information for interpreting cumulative effects. Unlike fish, aquatic insects are abundant in most small, first and second order streams. Individuals are relatively easy to identify to family level, and many "intolerant" taxa can be identified to lower taxonomic levels with ease.

Fish are good stream indicators over longer time periods and broad habitat conditions because they are relatively long-lived and mobile. Fish communities generally include a range of species that represents a variety of trophic levels (omnivores, herbivores, insectivores, planktivores, and piscivores). Fish tend to integrate the effects of lower trophic levels; thus, their community structure reflects the prevailing food sources and habitat conditions. Fish are relatively easy to collect and identify to the species level. Most specimens can be sorted and identified in the field by experienced fisheries scientists and subsequently released unharmed.

A review of the literature indicates that a wide variety of metrics are used to measure the aquatic insect and fish community. Community indices, such as the Index of Biotic Integrity (IBI) for fish and the Benthic Index of Biotic Integrity (B-IBI) for the aquatic insect community are a weighted combination of various metrics that typically characterize the community from "excellent" to "poor." Common metrics of aquatic community are often based on a composite of measures, such as species richness, abundance, tolerance, trophic status, and native status. Combined indices (C-IBI) measure both fish and aquatic insect metrics and a variety of physical habitat conditions to classify streams. Table 47 lists several common metrics used in stream assessments. It should be clearly noted that community and combined indices rely on different measurements and cannot be directly compared. For a comprehensive review of aquatic community indicators, see Barbour *et al.*(1999).

5.2.2 Watershed Development Indices

Watershed IC, housing density, population density, and percent urban land have all been used as indices of the degree of watershed development. In addition, reverse indicators such as percent forest cover and riparian continuity have also been used. The majority of studies so far have used IC to explore the relationship between urbanization and aquatic diversity. Percent urban land has been the second most frequently used indicator to describe the impact of watershed development. Table 48 compares the four watershed development indices and the thresholds where significant impacts to aquatic life are typically observed.

Table 47: Examples of Biodiversity Metrics Used to Assess Aquatic Communities		
Measurement	**Applied to:**	**Definition of Measurement**
Abundance	Fish, Aquatic Insects	Total number of individuals in a sample; sometimes modified to exclude tolerant species.
Taxa Richness	Fish, Aquatic Insects	Total number of unique taxa identified in a sample. Typically, an increase in taxa diversity indicates better water and habitat quality.
EPT Index	Aquatic Insects	Taxa belonging to the following three groups: *Ephemeroptera* (mayflies), *Plecoptera* (stoneflies), *Trichoptera* (caddisflies). Typically, species in these orders are considered to be pollution-intolerant taxa and are generally the first to disappear with stream quality degradation.
Native Status	Fish	Native vs. non-native taxa in the community.
Specific Habitat	Fish	Riffle benthic insectivorous individuals. Total number of benthic insectivores. Often these types of individuals, such as darters, sculpins, and dace are found in high velocity riffles and runs and are sensitive to physical habitat degradation.
		Minnow species Total number of minnow species present. Often used as an indicator of pool habitat quality. Includes all species present in the family Cyprinidae, such as daces, minnows, shiners, stonerollers, and chubs.
Tolerant Species	Fish, Aquatic Insects	The total number of species sensitive to and the number tolerant of degraded conditions. Typically, intolerant species decline with decreasing water quality and stream habitat. A common high pollution-tolerant species that is frequently used is Chironomids.
Dominance	Fish, Aquatic Insects	The proportion of individuals at each station from the single most abundant taxa at that particular station. Typically, a community dominated by a single taxa may be indicative of stream degradation.
Functional Feeding Group Comparisons	Fish	Omnivores/ Generalists: The proportion of individuals characterized as omnivores or generalists to the total number of individuals. Typically, there is a shift away from specialized feeding towards more opportunistic feeders under degraded conditions as food sources become unreliable.
		Insectivores: The proportion of individuals characterized as insectivores to the total number of individuals. Typically, the abundance of insectivores decreases relative to increasing stream degradation.
	Aquatic Insects	Others: The proportion of individuals characterized as shredders, scrapers, or filter feeders to the total number of individuals. Typically, changes in the proportion of functional feeders characterized as shredders can be reflective of contaminated leaf matter. In addition, an overabundance of scrapers over filterers can be indicative of increased benthic algae.
Disease/ Anomalies	Fish	Proportion of individuals with signs of disease or abnormalities. This is ascertained through gross external examination for abnormalities during the field identification process. Typically, this metric assumes that incidence of disease and deformities increases with increasing stream degradation.

** This table is not meant to provide a comprehensive listing of metrics used for diversity indices; it is intended to provide examples of types of measures used in biological stream assessments (see Barbour* et al., *1999).*

5.2.3 General Trends

Most research suggests that a decline in both species abundance and diversity begins at or around 10% watershed IC (Schueler, 1994a). However, considerable variations in aquatic diversity are frequently observed from five to 20% IC, due to historical alterations, the effectiveness of watershed management, prevailing riparian conditions, co-occurrence of stressors, and natural biological variation (see Chapter 1).

Figures 40 through 42 display the negative relationship commonly seen between biotic indices and various measures of watershed development. For example, stream research in the Maryland Piedmont indicated that IC was the best predictor of stream condition, based on a combined fish and aquatic insect IBI (MNCPPC, 2000). In general, streams with less than 6% watershed IC were in "excellent" condition, whereas streams in "good" condition had less than 12% IC, and streams in "fair" condition had less than 20%. Figure 40 shows the general boundaries and typical variation seen in MNCPPC stream research.

Figure 41 illustrates that B-IBI scores and Coho Salmon/Cutthroat Trout Ratio are a function of IC for 31 streams in Puget Sound, Washington. The interesting finding was that "good" to "excellent" B-IBI scores (greater than 25) were reported in watersheds that had less than 10% IC, with eight notable outliers. These outliers had greater IC (25 to 35%) but similar B-IBI scores. These outliers are unique in that they had a large upstream wetland and/or a large, intact riparian corridor upstream (i.e. >70% of stream corridor had buffer width >100 feet).

Figure 42 depicts the same negative relationship between watershed urbanization and fish-IBI scores but uses population density as the primary metric of development (Dreher, 1997). The six-county study area included the Chicago metro area and outlying rural watersheds. Significant declines in fish-IBI scores were noted when population density exceeded 1.5 persons per acre.

The actual level of watershed development at which an individual aquatic species begins to decline depends on several variables, but may be lower than that indicated by the ICM. Some researchers have detected impacts for individual aquatic species at watershed IC levels as low as 5%. Other research has suggested that the presence of certain stressors, such as sewage treatment plant discharges (Yoder and Miltner, 2000) or construction sites (Reice, 2000) may alter the ICM and lower the level of IC at which biodiversity impacts become evident.

Table 48: Alternate Land Use Indicators and Significant Impact Levels (Brown, 2000; Konrad and Booth, 2002)			
Land Use Indicator	**Level at which Significant Impact Observed**	**Typical Value for Low Density Residential Use**	**Comments**
% IC	10-20%	10%	Most accurate; highest level of effort and cost
Housing Density	>1 unit/acre	1 unit/acre	Low accuracy in areas of substantial commercial or industrial development; less accurate at small scales
Population Density	1.5 to 8+ people/acre	2.5 people/acre	Low accuracy in areas of substantial commercial or industrial development; less accurate at small scales
% Urban Land Use	33% (variable)	10-100%	Does not measure intensity of development; moderately accurate at larger watershed scales
Road Density	5 miles/square mile	2 miles/square mile	Appears to be a potentially useful indicator

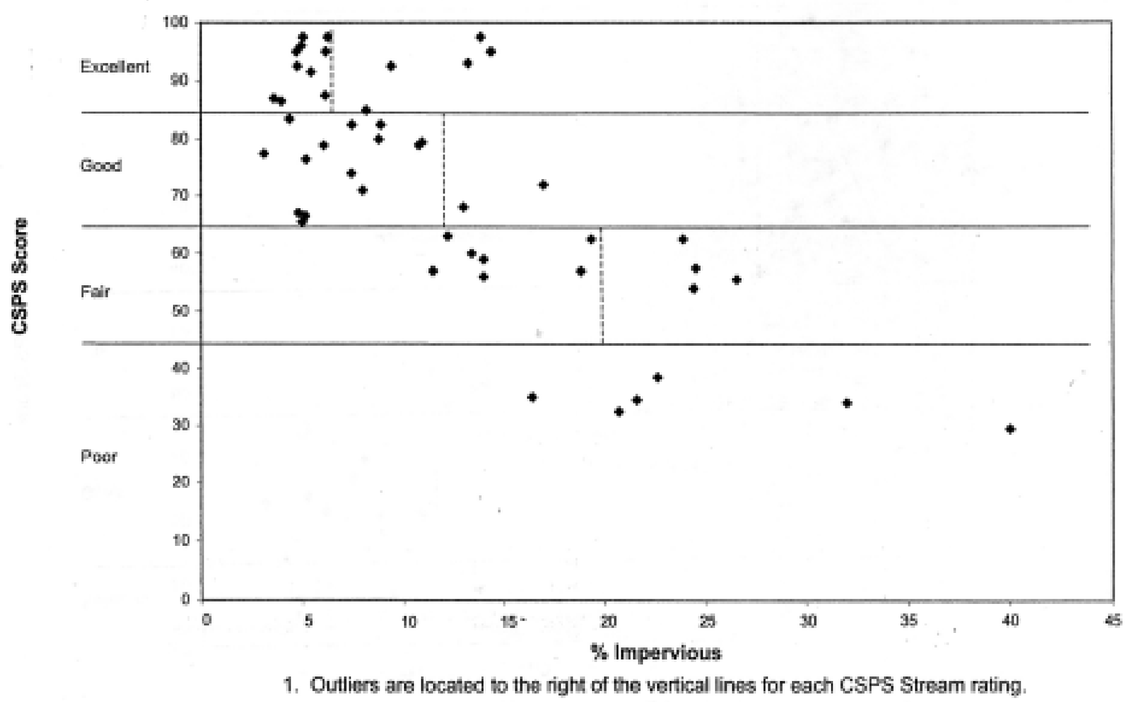


Figure 40: Combined Fish and Benthic IBI vs. IC in Maryland Piedmont Streams (MNCPPC, 2000)

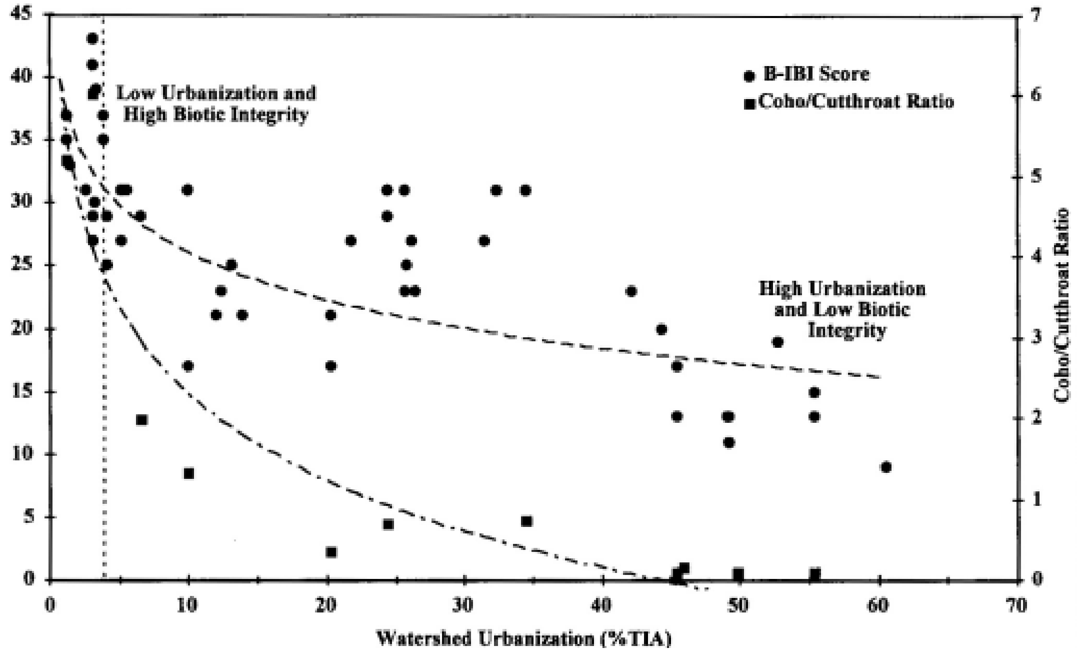


Figure 41: Relationship Between B-IBI, Coho/Cutthroat Ratios, and Watershed IC in Puget Sound Streams (Horner *et al.*, 1997)

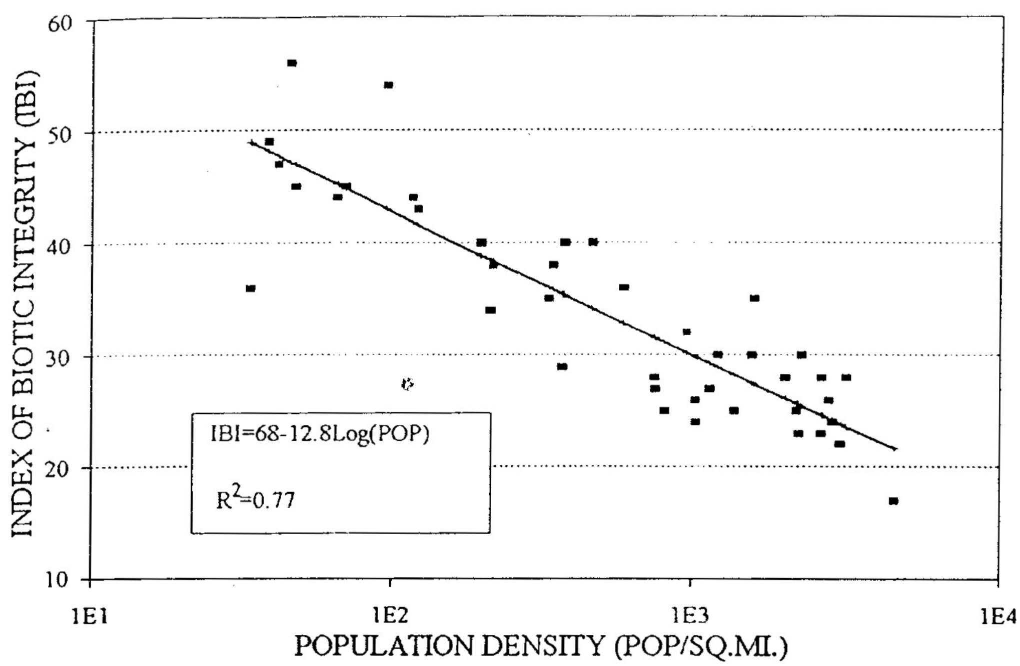


Figure 42: Index for Biological Integrity as a Function of Population Density in Illinois (Dreher, 1997)

5.3 Effects on Aquatic Insect Diversity

The diversity, richness and abundance of the aquatic insect community is frequently used to indicate urban stream quality. Aquatic insects are a useful indicator because they form the base of the stream food chain in most regions of the country. For this reason, declines or changes in aquatic insect diversity are often an early signal of biological impact due to watershed development. The aquatic insect community typically responds to increasing development by losing species diversity and richness and shifting to more pollution-tolerant species. More than 30 studies illustrate how IC and urbanization affect the aquatic insect community. These are summarized in Tables 49 and 50.

5.3.1 Findings Based on IC Indicators

Klein (1979) was one of the first researchers to note that aquatic insect diversity drops sharply in streams where watershed IC exceeded 10 to 15%. While "good" to "fair" diversity was noted in all headwater streams with less than 10% IC, nearly all streams with 12% or more watershed IC recorded "poor" diversity. Other studies have confirmed this general relationship between IC and the decline of aquatic insect species diversity. Their relationships have been an integral part in the development of the ICM. The sharp drop in aquatic insect diversity at or around 12 to 15% IC was also observed in streams in the coastal plain and Piedmont of Delaware (Maxted and Shaver, 1997).

Impacts at development thresholds lower than 10% IC have also been observed by Booth (2000), Davis (2001), Horner *et al.* (1997) and Morse (2001). There seems to be a general recognition that the high levels of variability observed below 10% IC indicate that other factors, such as riparian condition, effluent discharges, and pollution legacy may be better indicators of aquatic insect diversity (Horner and May, 1999; Kennen, 1999; Steedman, 1988; Yoder *et al.*, 1999).

The exact point at which aquatic insect diversity shifts from fair to poor is not known with absolute precision, but it is clear that few, if any, urban streams can support diverse aquatic insect communities with more than 25% IC. Indeed, several researchers failed to find aquatic insect communities with good or excellent diversity in any highly urban stream (Table 52). Indeed, MNCPPC (2000) reported that all streams with more than 20% watershed IC were rated as "poor."

Several good examples of the relationship between IC and B-IBI scores are shown in Figures 43 through 45. Figure 43 depicts the general trend line in aquatic insect diversity as IC increased at 138 stream sites in Northern Virginia (Fairfax County, 2001). The survey study concluded that stream degradation occurred at low levels of IC, and that older developments lacking more efficient site design and stormwater controls tended to have particularly degraded streams. Figures 44 and 45 show similar trends in the relationship between IC and aquatic insect B-IBI scores in Maryland and Washington streams. In particular, note the variability in B-IBI scores observed below 10% IC in both research studies.

Often, shift in the aquatic insect community from pollution-sensitive species to pollution-tolerant species occurs at relatively low IC levels (<10%). This shift is often tracked using the EPT metric, which evaluates sensitive species found in the urban stream community in the orders of *Ephemeroptera* (mayflies), *Plecoptera* (stoneflies), and *Trichoptera* (caddisflies). EPT species frequently disappear in urban streams and are replaced by more pollution-tolerant organisms, such as chironomids, tubificid worms, amphipods and snails.

In undisturbed streams, aquatic insects employ specialized feeding strategies, such as shredding leaf litter, filtering or collecting organic matter that flows by, or preying on other insects. These feeding guilds are greatly reduced in urban streams and are replaced by grazers, collectors and deposit feeders. Maxted and Shaver (1997) found that 90% of sensitive

Table 49: Recent Research Examining the Relationship Between IC and Aquatic Insect Diversity in Streams			
Index	**Key Finding (s)**	**Source**	**Location**
Community Index	Three years stream sampling across the state at 1000 sites found that when IC was >15%, stream health was never rated good based on a C-IBI.	Boward *et al.*, 1999	MD
Community Index	Insect community and habitat scores were all ranked as poor in five subwatersheds that were greater than 30% IC.	Black and Veatch, 1994	MD
Community Index	Puget sound study finds that some degradation of aquatic invertebrate diversity can occur at any level of human disturbance (at least as measured by IC). 65% of watershed forest cover usually indicates a healthy aquatic insect community.	Booth, 2000	WA
Community Index	In a Puget Sound study, the steepest decline of B-IBI was observed after 6% IC. There was a steady decline, with approximately 50% reduction in B-IBI at 45% IC.	Horner *et al.*, 1997	WA
Community Index	B-IBI decreases with increasing urbanization in study involving 209 sites, with a sharp decline at 10% IC. Riparian condition helps mitigate effects.	Steedman, 1988	Ontario
Community Index	Wetlands, forest cover and riparian integrity act to mitigate the impact of IC on aquatic insect communities.	Horner *et al.*, 2001	WA, MD, TX
Community Index	B-IBI declines for aquatic insect with increasing IC at more than 200 streams.	Fairfax Co., 2001	VA
Community Index	Two-year stream study of eight Piedmont watersheds reported B-IBI scores declined sharply at an IC threshold of 15-30%.	Meyer and Couch,2000	GA
Community Index	Montgomery County study; subwatersheds with <12% IC generally had streams in good to excellent condition based on a combined fish and aquatic insect IBI. Watersheds with >20% IC had streams in poor condition.	MNCPPC, 2000	MD
Community Index	Study of 1st, 2nd, and 3rd order streams in the Patapsco River Basin showed negative relationship between B-IBI and IC.	Dail *et al.*, 1998	MD
Community Index	While no specific threshold was observed, impacts were seen at even low levels of IC. B-IBI values declined with increasing IC, with high scores observed only in reaches with <5% IC or intact riparian zones or upstream wetlands.	Horner and May, 1999	WA
Community Index	The C-IBI also decreased by 50% at 10-15% IC. These trends were particularly strong at low-density urban sites (0-30% IC).	Maxted and Shaver, 1997	DE
Diversity	In both coastal plain and Piedmont streams, a sharp decline in aquatic insect diversity was found around 10-15% IC.	Shaver *et al.*, 1995	DE
Diversity	In a comparison of Anacostia subwatersheds, there was significant decline in the diversity of aquatic insects at 10% IC.	MWCOG, 1992	DC
Diversity	In several dozen Piedmont headwater streams, aquatic diversity declined significantly beyond 10-12% IC.	Klein, 1979	MD
EPT Value	In a 10 stream study with watershed IC ranging from three to 30%, a significant decline in EPT values was reported as IC increased (r2 = 0.76).	Davis, 2001	MO
Sensitive Species	In a study of 38 wadeable, non-tidal streams in the urban Piedmont, 90% of sensitive organisms were eliminated from the benthic community after watershed IC reaches 10-15%.	Maxted and Shaver, 1997	DE
Species Abundance EPT values	For streams draining 20 catchments across the state, an abrupt decline in species abundance and EPT taxa was observed at approximately 6% IC.	Morse, 2001	ME

Table 50: Recent Research Examining the Relationship of Other Indices of Watershed Development on Aquatic Insect Diversity in Streams

Biotic	Key Finding (s)	Source	Location
Percent Urban Land use			
Community Index	Study of 700 streams in 5 major drainage basins found that the amount of urban land and total flow of municipal effluent were the most significant factors in predicting severe impairment of the aquatic insect community. Amount of forested land in drainage area was inversely related to impairment severity.	Kennen, 1999	NJ
Community Index	All 40 urban sites sampled had fair to very poor B-IBI scores, compared to undeveloped reference sites.	Yoder, 1991	OH
Community Index	A negative correlation between B-IBI and urban land use was noted. Community characteristics show similar patterns between agricultural and forested areas the most severe degradation being in urban and suburban areas.	Meyer and Couch, 2000	GA
EPT Value, Diversity, Community Index	A comparison of three stream types found urban streams had lowest diversity and richness. Urban streams had substantially lower EPT scores (22% vs 5% as number of all taxa, 65% vs 10% as percent abundance) and IBI scores in the poor range.	Crawford and Lenat, 1989	NC
Sensitive Species	Urbanization associated with decline in sensitive taxa, such as mayflies, caddisflies and amphipods while showing increases in oligochaetes.	Pitt and Bozeman, 1982	CA
Sensitive Species	Dramatic changes in aquatic insect community were observed in most urbanizing stream sections. Changes include an abundance of pollution-tolerant aquatic insect species in urban streams.	Kemp and Spotila, 1997	PA
Diversity	As watershed development levels increased, the aquatic insect diversity declined.	Richards *et al.*, 1993	MN
Diversity	Significant negative relationship between number of aquatic insect species and degree of urbanization in 21 Atlanta streams.	Benke *et al.*, 1981	GA
Diversity	Drop in insect taxa from 13 to 4 was noted in urban streams.	Garie and McIntosh, 1986	NJ
Diversity	Aquatic insect taxa were found to be more abundant in non-urban reaches than in urban reaches of the watershed.	Pitt and Bozeman, 1982	CA
Diversity	A study of five urban streams found that as watershed land use shifted from rural to urban, aquatic insect diversity decreased.	Masterson and Bannerman, 1994	WI
Other Land Use Indicators			
Community Index	Most degraded streams were found in developed areas, particularly older developments lacking newer and more efficient stormwater controls.	Fairfax Co., 2001	VA
Diversity	Urban streams had sharply lower aquatic insect diversity with human population above four persons/acre in northern VA.	Jones and Clark, 1987	VA
EPT Value	Monitoring of four construction sites in three varying regulatory settings found that EPT richness was related to enforcement of erosion and sediment controls. The pattern demonstrated that EPT richness was negatively affected as one moved from upstream to at the site, except for one site.	Reice, 2000	NC
Sensitive Species	In a Seattle study, aquatic insect community shifted to chironomid, oligochaetes and amphipod species that are pollution-tolerant and have simple feeding guild.	Pedersen and Perkins,1986	WA

species (based on EPT richness, % EPT abundance, and Hilsenhoff Biotic Index) were eliminated from the aquatic insect community when IC exceeded 10 to 15% in contributing watersheds of Delaware streams (Figure 46). In a recent study of 30 Maine watersheds, Morse (2001) found that reference streams with less than 5% watershed IC had significantly more EPT taxa than more urban streams. He also observed no significant differences in EPT Index values among streams with six to 27% watershed IC (Figure 47).

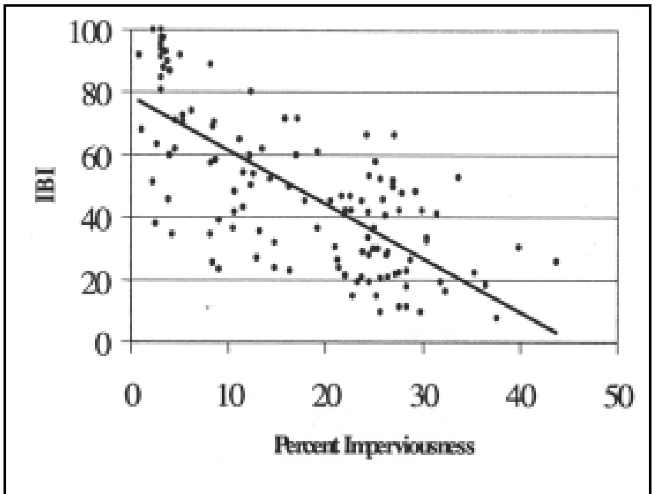


Figure 43: Trend Line Indicating Decline in Benthic IBI as IC Increases in Northern VA Streams (Fairfax County, 2001)

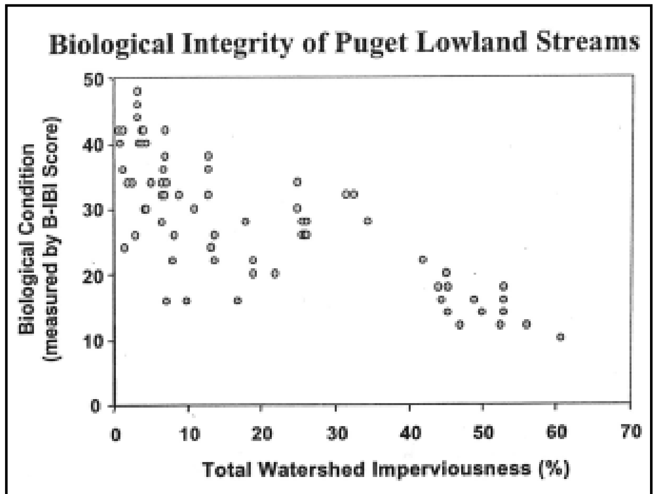


Figure 44: Relationship Between IC and B-IBI Scores in Aquatic Insects in Streams of the Puget Sound Lowlands (Booth, 2000)

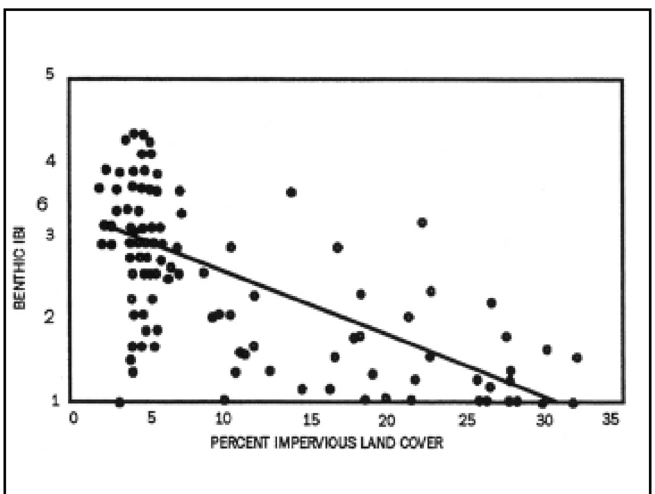


Figure 45: IC and B-IBI at Stream Sites in the Patapsco River Basin, MD (Dail *et al.*, 1998)

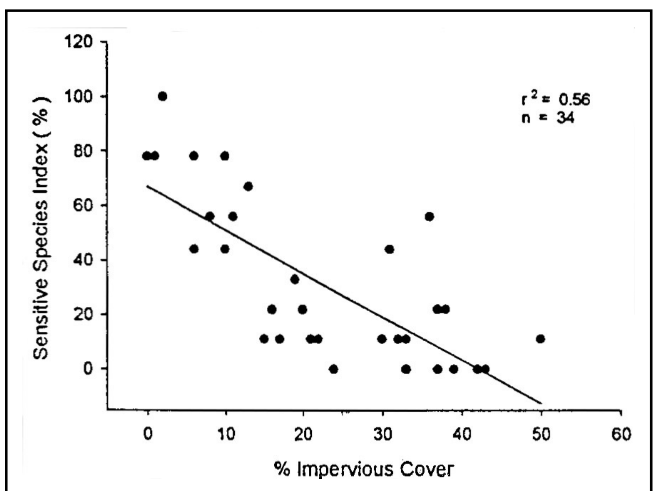


Figure 46: IC vs. Aquatic Insect Sensitivity - EPT Scores in Delaware Streams (Maxted and Shaver, 1997)

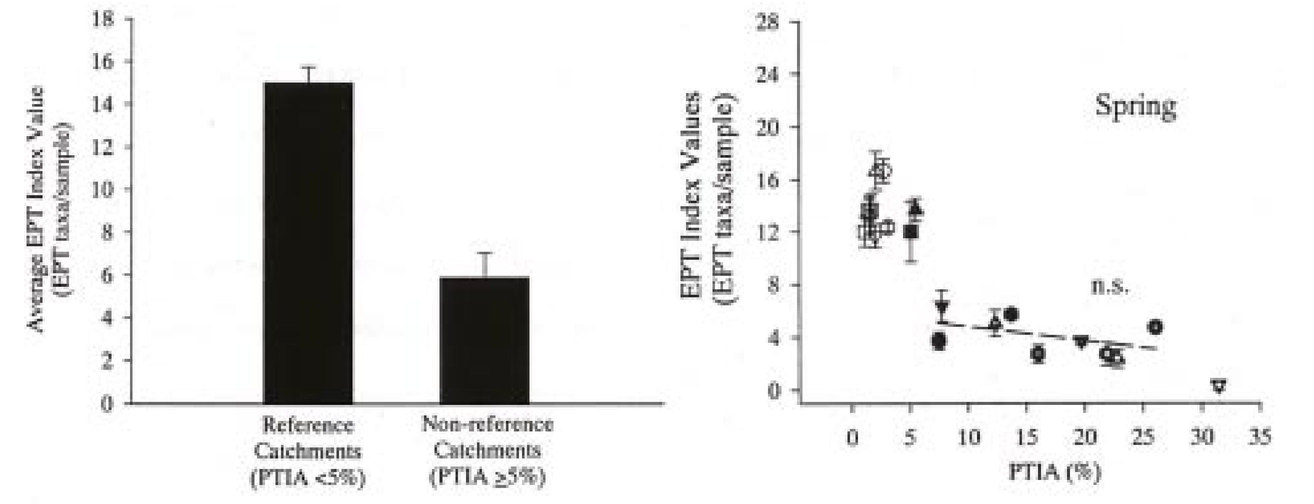


Figure 47: Average and Spring EPT Index Values vs.% IC in 20 Small Watersheds in Maine (Morse, 2001)

5.3.2 Findings Based on Other Development Indicators

Development indices, such as percent urban land use, population density, and forest and riparian cover have also been correlated with changes in aquatic insect communities in urban streams. Declines in benthic IBI scores have frequently been observed in proportion to the percent urban land use in small watersheds (Garie and McIntosh, 1986; Kemp and Spotila, 1997; Kennen, 1999; Masterson and Bannerman, 1994; Richards *et al.*, 1993; USEPA, 1982).

A study in Washington state compared a heavily urbanized stream to a stream with limited watershed development and found that the diversity of the aquatic insect community declined from 13 taxa in reference streams to five taxa in more urbanized streams (Pedersen and Perkins, 1986). The aquatic insect taxa that were lost were poorly suited to handle the variable erosional and depositional conditions found in urban streams. Similarly, a comparison of three North Carolina streams with different watershed land uses concluded the urban watershed had the least taxa and lowest EPT scores and greatest proportion of pollution-tolerant species (Crawford and Lenat, 1989).

Jones and Clark (1987) monitored 22 streams in Northern Virginia and concluded that aquatic insect diversity diminished markedly once watershed population density exceeded four or more people per acre. The population density roughly translates to ½ - 1 acre lot residential use, or about 10 to 20 % IC. Kennen (1999) evaluated 700 New Jersey streams and concluded that the percentage of watershed forest was positively correlated with aquatic insect density. Meyer and Couch (2000) reported a similar cover relationship between aquatic insect diversity and watershed and riparian forest cover for streams in the Atlanta, GA region. A study in the Puget Sound region found that aquatic insect diversity declined in streams once forest cover fell below 65% (Booth, 2000).

5.4 Effects on Fish Diversity

Fish communities are also excellent environmental indicators of stream health. In general, an increase in watershed IC produces the same kind of impact on fish diversity as it does for aquatic insects. The reduction in fish diversity is typified by a reduction in total species, loss of sensitive species, a shift toward more pollution-tolerant species, and decreased survival of eggs and larvae. More than 30 studies have examined the relationship between watershed development and fish diversity; they are summarized in Tables 51 and 52. About half of the research studies used IC as the major index of watershed development, while the remainder used other indices, such as percent urban land use, population density, housing density, and forest cover.

5.4.1 Findings Based on IC Indicators

Recent stream research shows a consistent, negative relationship between watershed development and various measures of fish diversity, such as diversity metrics, species loss and structural changes.

Typically, a notable decline in fish diversity occurs around 10 to 15% watershed IC (Boward *et al.*, 1999; Galli, 1994; Klein, 1979; Limburg and Schmidt, 1990; MNCPPC, 2000; MWCOG, 1992; Steward, 1983). A somewhat higher threshold was observed by Meyer and Couch (2000) for Atlanta streams with 15 to 30% IC; lower thresholds have also been observed (Horner *et al.*, 1997 and May *et al.*, 1997). A typical relationship between watershed IC and fish diversity is portrayed in Figure 48, which shows data from streams in the Patapsco River Basin in Maryland (Dail *et al.*, 1998). Once again, note the variability in fish-IBI scores observed below 10% IC.

Wang *et al.* (1997) evaluated 47 Wisconsin streams and found an apparent threshold around 10% IC. Fish-IBI scores were "good" to "excellent" below this threshold, but were consistently rated as "fair" to "poor." Additionally, Wang documented that the total number of fish species drops sharply when IC increases (Figure 49). Often, researchers also reported that increases in IC were strongly correlated with several fish metrics, such as increases in non-native and pollution-tolerant species in streams in Santa Clara, California (EOA, Inc., 2001).

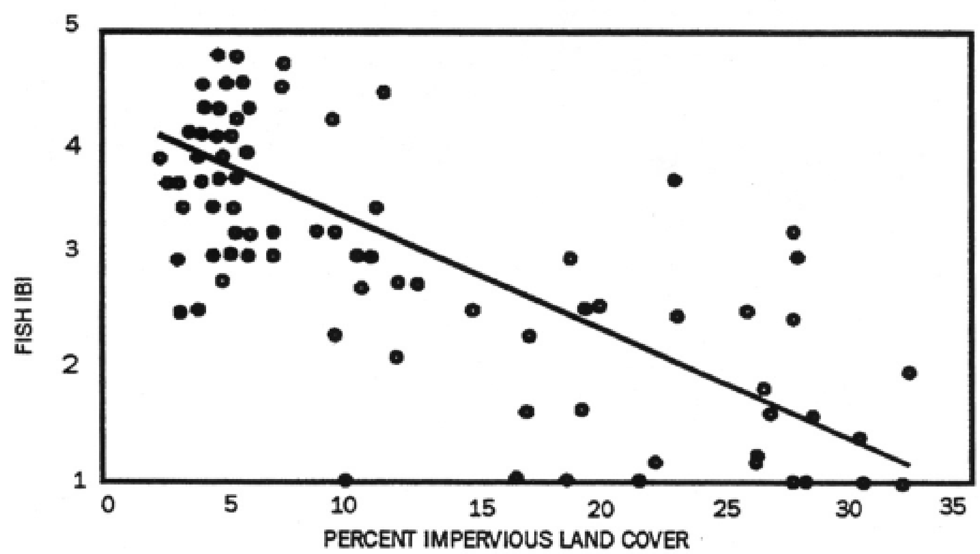


Figure 48: Fish-IBI vs. Watershed IC for Streams in the Patapsco River Basin, MD (Dail *et al.*, 1998)

Table 51: Recent Research Examining the Relationship Between Watershed IC and the Fish Community			
Biotic	**Key Finding (s)**	**Source**	**Location**
Abundance	Brown trout abundance and recruitment declined sharply at 10-15% IC.	Galli, 1994	MD
Salmonids	Seattle study showed marked reduction in coho salmon populations noted at 10-15% IC at nine streams.	Steward, 1983	WA
Anadromous Fish Eggs	Resident and anadromous fish eggs and larvae declined in 16 subwatersheds draining to the Hudson River with >10% IC area.	Limburg and Schmidt, 1990	NY
Community Index	1st, 2nd, and 3rd order streams in the Patapsco River Basin showed negative relationship between IBI and IC.	Dail *et al.*, 1998	MD
Community Index	Fish IBI and habitat scores were all ranked as poor in five subwatersheds that were greater than 30% IC.	Black and Veatch, 1994	MD
Community Index	In the Potomac subregion, subwatersheds with < 12% IC generally had streams in good to excellent condition based on a combined fish and aquatic insect IBI. Watersheds with >20% IC had streams in poor condition.	MNCPPC, 2000	MD
Community Index	In a two-year study of Piedmont streams draining eight watersheds representing various land uses in Chattahochee River Basin, fish community quality dropped sharply at an IC threshold of 15-30%.	Meyer and Couch, 2000	GA
Diversity	Of 23 headwater stream stations, all draining <10% IC areas, rated as good to fair; all with >12% were rated as poor. Fish diversity declined sharply with increasing IC between 10-12%.	Schueler and Galli, 1992	MD
Diversity, Sensitive Species	Comparison of 4 similar subwatersheds in Piedmont streams, there was significant decline in the diversity of fish at 10% IC. Sensitive species (trout and sculpin) were lost at 10-12%.	MWCOG, 1992	MD
Diversity, Community Index	In a comparison of watershed land use and fish community data for 47 streams between the 1970s and 1990s, a strong negative correlation was found between number species and IBI scores with effective connected IC. A threshold of 10% IC was observed with community quality highly variable below 10% but consistently low above 10% IC.	Wang *et al.*, 1997	WI
Diversity	In several dozen Piedmont headwater streams fish diversity declined significantly in areas beyond 10-12% IC.	Klein, 1979	MD
Diversity , Abundance, Non-native Species	IC strongly associated with several fisheries species and individual-level metrics, including number of pollution-tolerant species, diseased individuals, native and non-native species and total species present	EOA, Inc., 2001	CA
Juvenile Salmon Ratios	In Puget Sound study, the steepest decline of biological functioning was observed after six percent IC. There was a steady decline, with approximately 50% reduction in initial biotic integrity at 45% IC area.	Horner *et al.*, 1997	WA
Juvenile Salmon Ratio	Physical and biological stream indicators declined most rapidly during the initial phase of the urbanization process as total IC area exceeded the five to 10% range.	May *et al.*, 1997	WA
Salmonoid	Negative effects of urbanization (IC) with the defacto loss of non-structural BMPs (wetland forest cover and riparian integrity) on salmon ratios	Horner *et al.*, 2001	WA, MD, TX
Salmonoid, Sensitive Species	While no specific threshold was observed (impacts seen at even low levels of IC), Coho/cutthroat salmon ratios >2:1 were found when IC was < 5%. Ratios fell below one at IC levels below 20 %.	Horner and May, 1999	WA
Sensitive species, Salmonid	Three years stream sampling across the state (approximately 1000 sites), MBSS found that when IC was >15%, stream health was never rated good based on CBI, and pollution sensitive brook trout were never found in streams with >2% IC.	Boward *et al.*, 1999	MD
Sensitive Species, Salmonids	Seattle study observed shift from less tolerant coho salmon to more tolerant cutthroat trout population between 10 and 15% IC at nine sites.	Luchetti and Feurstenburg 1993	WA

Sensitive fish are defined as species that strongly depend on clean and stable bottom substrates for feeding and/or spawning. Sensitive fish often show a precipitous decline in urban streams. The loss of sensitive fish species and a shift in community structure towards more pollution-tolerant species is confirmed by multiple studies. Figure 50 shows the results of a comparison of four similar subwatersheds in the Maryland Piedmont that were sampled for the number of fish species present (MWCOG, 1992). As the level of watershed IC increased, the number of fish species collected dropped. Two sensitive species, including sculpin, were lost when IC increased from 10 to 12%, and four more species were lost when IC reached 25%. Significantly, only two species remained in the fish community at 55% watershed IC.

Salmonid fish species (trout and salmon) and anadromous fish species appear to be particularly impacted by watershed IC. In a study in the Pacific Northwest, sensitive coho salmon were seldom found in watersheds above 10 or 15% IC (Luchetti and Feurstenburg, 1993 and Steward, 1983). Key stressors in urban streams, such as higher peak flows, lower dry weather flows, and reduction in habitat complexity (e.g. fewer pools, LWD, and hiding places) are believed to change salmon species composition, favoring cutthroat trout populations over the natural coho populations (WDFW, 1997).

A series of studies from the Puget Sound reported changes in the coho/cutthroat ratios of juvenile salmon as watershed IC increased (Figure 51). Horner *et al.* (1999) found Coho/Cutthroat ratios greater than 2:1 in watersheds with less than 5 % IC. Ratios fell below 1:1 when IC exceeded 20%. Similar results were reported by May *et al.* (1997). In the mid-Atlantic region, native trout have stringent temperature and habitat requirements and are seldom present in watersheds where IC exceeds 15% (Schueler, 1994a). Declines in trout spawning success are evident above 10% IC. In a study of over 1,000 Maryland streams, Boward *et al.* (1999) found that sensitive brook trout were never found in streams that had more than 4% IC in their contributing watersheds.

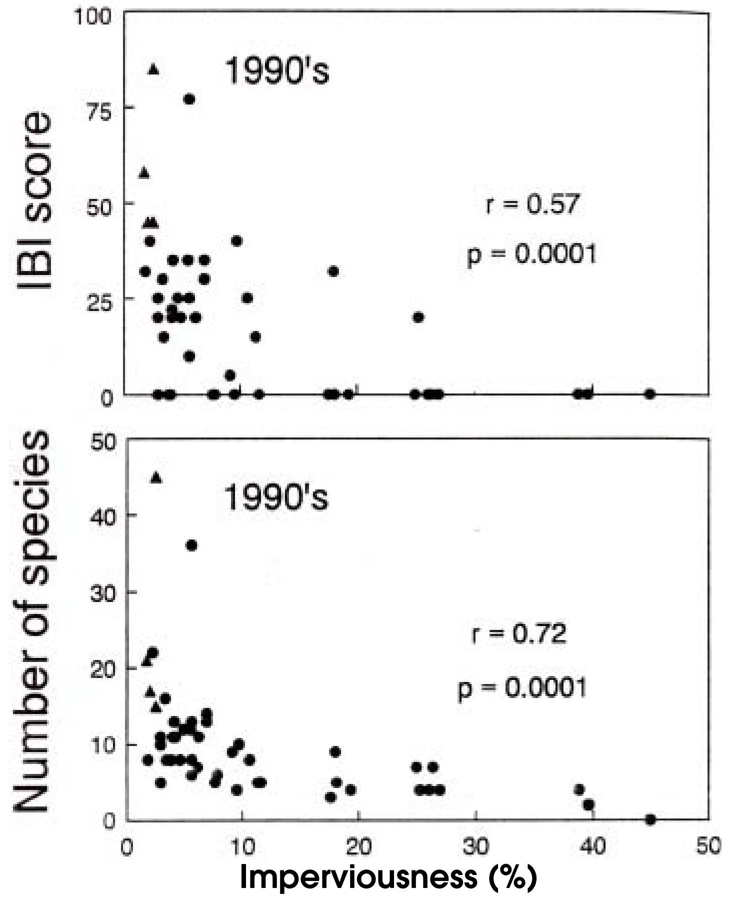


Figure 49: Fish-IBI and Number of Species vs. % IC in Wisconsin Streams (Wang *et al.*, 1997)

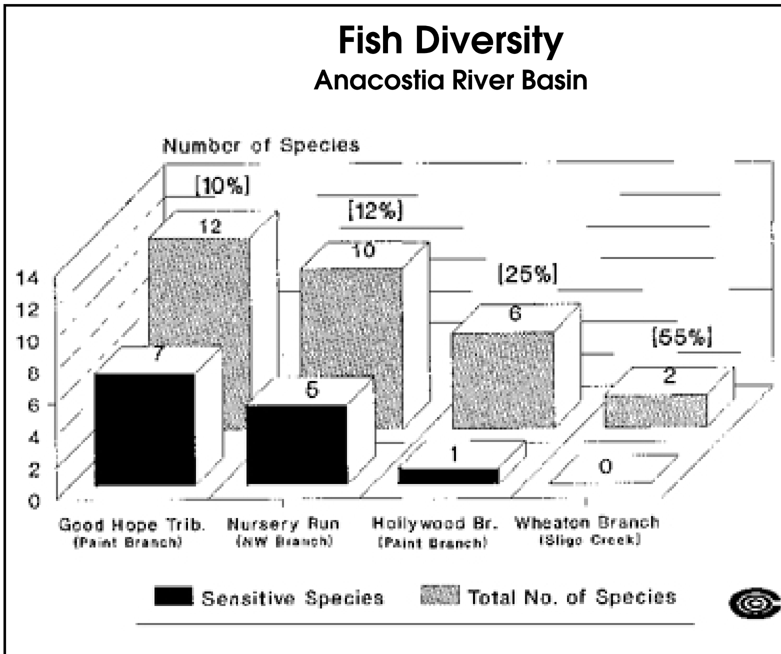


Figure 50: IC and Effects on Fish Species Diversity in Four Maryland Subwatersheds (MWCOG, 1992)

Table 52: Recent Research Examining Urbanization and Freshwater Fish Community Indicators

Biotic	Key Finding (s)	Source	Location
Urbanization			
Community Index	All 40 urban sites sampled had fair to very poor IBI scores, compared to undeveloped reference sites.	Yoder, 1991	OH
Community Index	Negative correlations between biotic community and riparian conditions and forested areas were found. Similar levels of fish degradation were found between suburban and agricultural; urban areas were the most severe.	Meyer and Couch, 2000	GA
Community Index	Residential urban land use caused significant decrease in fish-IBI scores at 33%. In more urbanized Cuyahoga, a significant drop in IBI scores occurred around 8% urban land use in the watershed. When watersheds smaller than 100mi^2 were analyzed separately, the level of urban land associated with a significant drop in IBI scores occurred at around 15%. Above one du/ac, most sites failed to attain biocriteria regardless of degree of urbanization.	Yoder *et al.*, 1999	OH
Community Index, Abundance	As watershed development increased to about 10%, fish communities simplified to more habitat and trophic generalists and fish abundance and species richness declined. IBI scores for the urbanized stream fell from the good to fair category.	Weaver, 1991	VA
Diversity	A study of five urban streams found that as land use shifted from rural to urban, fish diversity decreased.	Masterson and Bannerman, 1994	WI
Diversity, Community Index	A comparison of three stream types found urban streams had lowest diversity and richness. Urban streams had IBI scores in the poor range.	Crawford and Lenat, 1989	NC
Salmon Spawning, Flooding Frequency	In comparing three streams over a 25-year period (two urbanizing and one remaining forested), increases in flooding frequencies and decreased trends in salmon spawning were observed in the two urbanizing streams, while no changes in flooding or spawning were seen in the forested system.	Moscript and Montgomery, 1997	WA
Sensitive Species	Observed dramatic changes in fish communities in most urbanizing stream sections, such as absence of brown trout and abundance of pollution-tolerant species in urban reaches.	Kemp and Spotila, 1997	PA
Sensitive Species, Diversity	Decline in sensitive species diversity and composition and changes in trophic structure from specialized feeders to generalists was seen in an urbanizing watershed from 1958 to 1990. Low intensity development was found to affect warm water stream fish communities similarly as more intense development.	Weaver and Garman, 1994	VA
Warm Water Habitat Biocriteria	25-30% urban land use defined as the upper threshold where attainment of warm water habitat biocriterion is effectively lost. Non-attainment also may occur at lower thresholds given the co-occurrence of stressors, such as pollution legacy, WTPs and CSOs.	Yoder and Miltner, 2000	OH
Community Index, Habitat	The amount of urban land use upstream of sample sites had a strong negative relationship with biotic integrity, and there appeared to be a threshold between 10 and 20% urban land use where IBI scores declined dramatically. Watersheds above 20% urban land invariably had scores less than 30 (poor to very poor). Habitat scores were not tightly correlated with degraded fish community attributes.	Wang *et al.*, 1997	WI
Community Index	A study in the Patapsco Basin found significant correlation of fish IBI scores with percent urbanized land over all scales (catchment, riparian area, and local area).	Roth *et al.*, 1998	MD

Table 52 (continued): Recent Research Examining Urbanization and Freshwater Fish Community Indicators			
Biotic	**Key Finding (s)**	**Source**	**Location**
	Urbanization		
Sensitive Species	Evaluated effects of runoff in both urban and non-urban streams; found that native species dominated the non-urban portion of the watershed but accounted for only seven percent of species found in the urban portions of the watershed.	Pitt, 1982	CA
	Other Land Use Indicators		
Community Index, Habitat	Atlanta study found that as watershed population density increased, there was a negative impact on urban fish and habitat. Urban stream IBI scores were inversely related to watershed population density, and once density exceeded four persons/acre, urban streams were consistently rated as very poor.	Couch *et al.*, 1997	GA
Community Index	In an Atlanta stream study, modified IBI scores declined once watershed population density exceeds four persons/acre in 21 urban watersheds	DeVivo *et al.*, 1997	GA
Community Index	In a six-county study (including Chicago, its suburbs and outlying rural/agricultural areas), streams showed a strong correlation between population density and fish community assessments such that as population density increased, community assessment scores went from the better - good range to fair - poor. Significant impacts seen at 1.5 people/acre.	Dreher, 1997	IL
Community Index	Similarly, negative correlations between biotic community and riparian conditions and forested areas were also found. Similar levels of fish degradation were found between suburban and agricultural; urban areas were the most severe.	Meyer and Couch, 2000	GA
Community Index	Amount of_forested land in basin directly related to IBI scores for fish community condition.	Roth *et al.*, 1996	MD
Salmonid, Sensitive Species	Species community changes from natural coho salmon to cutthroat trout population with increases in peak flow, lower low flow, and reductions in stream complexity.	WDFW, 1997	WA

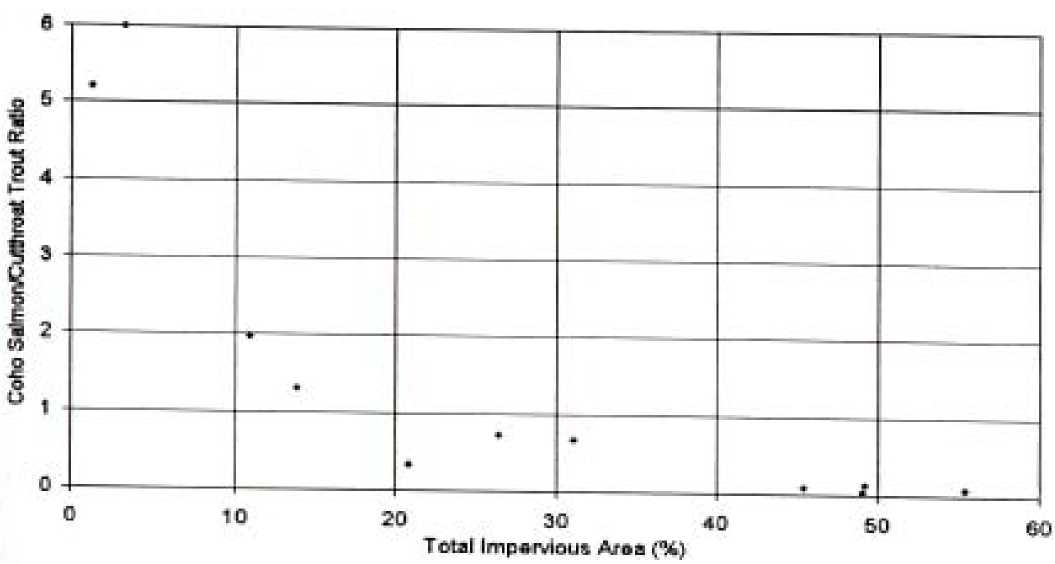


Figure 51: Coho Salmon/Cutthroat Trout Ratio for Puget Sound Streams (Horner *et al.*, 1997)

Many fish species have poor spawning success in urban streams and poor survival of fish eggs and fry. Fish barriers, low intragravel dissolved oxygen, sediment deposition and scour are all factors that can diminish the ability of fish species to successfully reproduce. For example, Limburg and Schmidt (1990) discovered that the density of anadromous fish eggs and larvae declined sharply in subwatersheds with more than 10% IC.

5.4.2 Findings Based on Other Development Indicators

Urban land use has frequently been used as a development indicator to evaluate the impact on fish diversity. Streams in urban watersheds typically had lower fish species diversity and richness than streams located in less developed watersheds. Declines in fish diversity as a function of urban land cover have been documented in numerous studies (Crawford and Lenat, 1989; Masterson and Bannerman, 1994; Roth *et al., 1998;* Yoder, 1991, and Yoder *et al.*, 1999). USEPA (1982) found that native fish species dominated the fish community of non-urban streams, but accounted for only 7% of the fish community found in urban streams. Kemp and Spotila (1997) evaluated streams in Pennsylvania and noted the loss of sensitive species (e.g. brown trout) and the increase of pollution-tolerant species, such as sunfish and creek chub (Figure 52).

Wang *et al.* (1997) cited percentage of urban land in Wisconsin watersheds as a strong negative factor influencing fish-IBI scores in streams and observed strong declines in IBI scores with 10 to 20% urban land use. Weaver and Garman (1994) compared the historical changes in the warm-water fish community of a Virginia stream that had undergone significant urbanization and found that many of the sensitive species present in 1958 were either absent or had dropped sharply in abundance when the watershed was sampled in 1990. Overall abundance had dropped from 2,056 fish collected in 1958 to 417 in 1990. In addition, the 1990 study showed that 67% of the catch was bluegill and common shiner, two species that are habitat and trophic "generalists." This shift in community to more habitat and trophic generalists was observed at 10% urban land use (Weaver, 1991).

Yoder *et al.* (1999) evaluated a series of streams in Ohio and reported a strong decrease in warm-water fish community scores around 33% residential urban land use. In the more urbanized Cuyahoga streams, sharp drops in

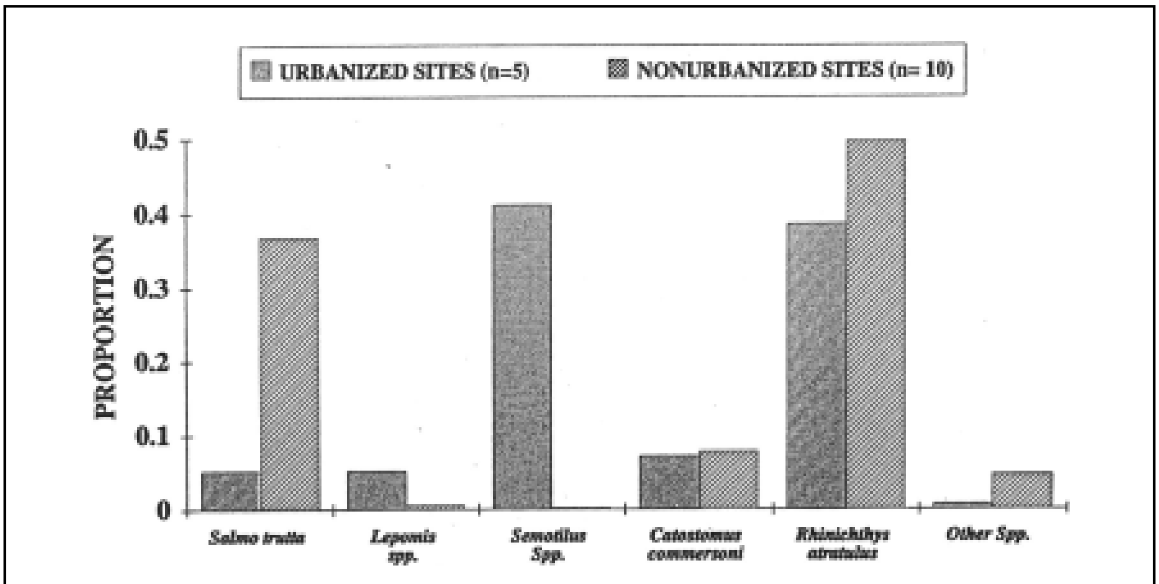


Figure 52: Mean Proportion of Fish Taxa in Urban and Non-Urban Streams, Valley Forge Watershed, PA (Kemp and Spotila, 1997)

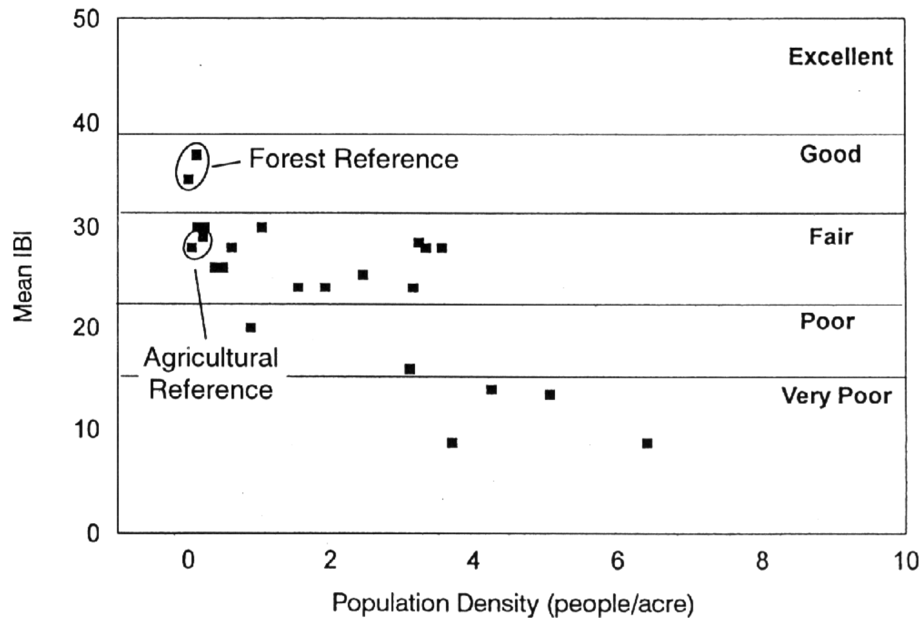


Figure 53: Relationship Between Watershed Population Density and Stream IBI Scores in Georgia Streams (DeVivo *et al.*, 1997)

fish-IBI scores occurred around 8% urban land use, primarily due to certain stressors which functioned to lower the non-attainment threshold. When watersheds smaller than 100mi^2 were analyzed separately, the percentage of urban land use associated with a sharp drop in fish-IBI scores was around 15%. In a later study, Yoder and Miltner (2000) described an upper threshold for quality warm-water fish habitat at 25 to 30% urban land use.

Watershed population and housing density have also been used as indicators of the health of the fish community. In a study of 21 urban watersheds in Atlanta, DeVivo *et al.* (1997) observed a shift in mean fish-IBI scores from "good to fair" to "very poor" when watershed population density exceeded four people/acre (Figure 53). A study of Midwest streams in metropolitan Illinois also found a negative relationship between increase in population density and fish communities, with significant impacts detected at population densities of 1.5 people or greater per acre (Dreher, 1997). In the Columbus and Cuyahoga watersheds in Ohio, Yoder *et al.* (1999) concluded that most streams failed to attain fish biocriteria above one dwelling unit/acre.

5.5 Effects on Amphibian Diversity

Amphibians spend portions of their life cycle in aquatic systems and are frequently found within riparian, wetland or littoral areas. Relatively little research has been conducted to directly quantify the effects of watershed development on amphibian diversity. Intuitively, it would appear that the same stressors that affect fish and aquatic insects would also affect amphibian species, along with riparian wetland alteration. We located four research studies on the impacts of watershed urbanization on amphibian populations; only one was related to streams (Boward *et al.,* 1999), while others were related to wetlands (Table 53).

A primary factor influencing amphibian diversity appears to be water level fluctuations (WLF) in urban wetlands that occur as a result of increased stormwater discharges. Chin (1996) hypothesized that increased WLF and other hydrologic factors affected the abundance of egg clutches and available amphibian breeding habitat, thereby ultimately influencing amphibian richness. Increased WLF can limit reproductive success by eliminating mating habitat and the emergent vegetation to which amphibians attach their eggs.

Taylor (1993) examined the effect of watershed development on 19 freshwater wetlands in King County, WA and concluded that the additional stormwater contributed to greater annual WLF. When annual WLF exceeded about eight inches, the richness of both the wetland plant and amphibian communities dropped sharply. Large increases in WLF were consistently observed in freshwater wetlands when IC in upstream watersheds exceeded 10 to 15%. Further research on streams and wetlands in the Pacific northwest by Horner *et al.* (1997) demonstrated the correlation between watershed IC and diversity of amphibian species. Figure 54 illustrates the relationship between amphibian species abundance and watershed IC, as documented in the study.

Table 53: Recent Research on the Relationship Between Percent Watershed Urbanization and the Amphibian Community

Indicator	Key Finding(s)	Reference Year	Location
% IC			
Reptile and Amphibian Abundance	In a three-year stream sampling across the state (approximately 1000 sites), MBSS found only hardy pollution-tolerant reptiles and amphibians in stream corridors with >25% IC drainage area.	Boward *et al.*, 1999	MD
Amphibian Density	Mean annual water fluctuation inversely correlated to amphibian density in urban wetlands. Declines noted beyond 10% IC.	Taylor, 1993	WA
Other Studies			
Species Richness	In 30 wetlands, species richness of reptiles and amphibians was significantly related to density of paved roads on lands within a two kilometer radius.	Findlay and Houlahan, 1997	Ontario
Species Richness	Decline in amphibian species richness as wetland WLF increased. While more of a continuous decline rather than a threshold, WLF = 22 centimeters may represent a tolerance boundary for amphibian community.	Horner *et al.*, 1997	WA
Amphibian Density	Mean annual water fluctuation inversely correlated to amphibian density in urban wetlands.	Taylor, 1993	WA

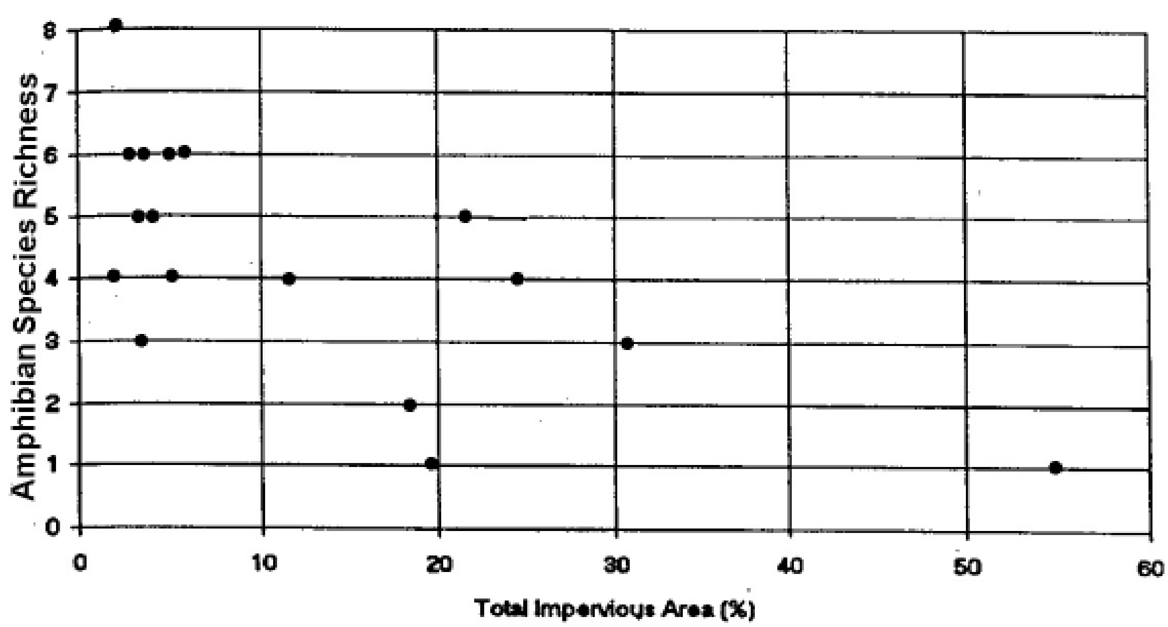


Figure 54: Amphibian Species Richness as a Function of Watershed IC in Puget Sound Lowland Wetlands (Horner *et al.*, 1997)

5.6 Effects on Wetland Diversity

We found a limited number of studies that evaluated the impact of watershed urbanization on wetland plant diversity (Table 54). Two studies used IC as an index of watershed development and observed reduced wetland plant diversity around or below 10% IC (Hicks and Larson, 1997 and Taylor, 1993). WLF and road density were also used as indicators (Findlay and Houlahan, 1997; Horner *et al.*, 1997; Taylor, 1993).

Horner *et al.* (1997) reported a decline in plant species richness in emergent and scrub-shrub wetland zones of the Puget Sound region as WLF increased. They cautioned that species numbers showed a continuous decline rather than a threshold value; however, it was indicated that WLF as small as 10 inches can represent a tolerance boundary for wetland plant communities. Horner further stated that in 90% of the cases where WLF exceeded 10 inches, watershed IC exceeded 21%.

Table 54: Recent Research Examining the Relationship Between Watershed Development and Urban Wetlands

Watershed Indicator / Biotic	Key Finding(s)	Reference	Location
% IC			
Insect Community	Significant declines in various indicators of wetland aquatic macro-invertebrate community health were observed as IC increased to 8-9%.	Hicks and Larson, 1997	CT
WLF, Water Quality	There is a significant increase in WLF, conductivity, fecal coliform bacteria, and total phosphorus in urban wetland as IC exceeds 3.5%.	Taylor *et al.*, 1995	WA
Plant Density	Declines in urban wetland plant density noted in areas beyond 10% IC.	Taylor, 1993	WA
Other Watershed Indicators			
Plant Density	Mean annual water fluctuation inversely correlated to plant density in urban wetlands.	Taylor, 1993	WA
Plant Species Richness	Decline in plant species richness in emergent and scrub-shrub wetland zones as WLF increased. While more of a continuous decline, rather than a threshold, WLF=22 centimeters may represent a tolerance boundary for the community	Horner *et al.*, 1997	WA
Plant Species Richness	In 30 wetlands, species richness was significantly related to density of paved roads within a two kilometer radius of the wetland. Model predicted that a road density of 2kilometers per hectare in paved road within 1000 meters of wetland will lead to a 13% decrease in wetland plant species richness.	Findlay and Houlahan,1997	Ontario

5.7 Effects on Freshwater Mussel Diversity

Freshwater mussels are excellent indicators of stream quality since they are filter-feeders and essentially immobile. The percentage of imperiled mussel species in freshwater ecoregions is high (Williams *et al.*, 1993). Of the 297 native mussel species in the United States, 72% are considered endangered, threatened, or of special concern, including 21 mussel species that are presumed to be extinct. Seventy mussel species (24%) are considered to have stable populations, although many of these have declined in abundance and distribution. Modification of aquatic habitats and sedimentation are the primary reasons cited for the decline of freshwater mussels (Williams *et al.,* 1993).

Freshwater mussels are very susceptible to smothering by sediment deposition. Consequently, increases in watershed development and sediment loading are suspected to be a factor leading to reduced mussel diversity. At sublethal levels, silt interferes with feeding and metabolism of mussels in general (Aldridge *et al.*, 1987). Major sources of mortality and loss of diversity in mussels include impoundment of rivers and streams, and eutrophication (Bauer, 1988). Changes in fish diversity and abundance due to dams and impoundments can also influence the availability of mussel hosts (Williams *et al.*, 1992).

Freshwater mussels are particularly sensitive to heavy metals and pesticides (Keller and Zam, 1991). Although the effects of metals and pesticides vary from one species to another, sub-lethal levels of PCBs, DDT, Malathion, Rotenone and other compounds are generally known to inhibit respiratory efficiency and accumulate in tissues (Watters, 1996). Mussels are more sensitive to pesticides than many other animals tested and often act as “first-alerts” to toxicity long before they are seen in other organisms.

We were unable to find any empirical studies relating impacts of IC on the freshwater mussel communities of streams.

5.8 Conclusion

The scientific record is quite strong with respect to the impact of watershed urbanization on the integrity and diversity of aquatic communities. We reviewed 35 studies that indicated that increased watershed development led to declines in aquatic insect diversity and about 30 studies showing a similar impact on fish diversity. The scientific literature generally shows that aquatic insect and freshwater fish diversity declines at fairly low levels of IC (10 to 15%), urban land use (33%), population density (1.5 to eight people/acre) and housing density (>1 du/ac). Many studies also suggest that sensitive elements of the aquatic community are affected at even lower levels of IC. Other impacts include loss of sensitive species and reduced abundance and spawning success. Research supports the ICM, although additional research is needed to establish the upper threshold at which watershed development aquatic biodiversity can be restored.

One area where more research is needed involves determining how regional and climatic variations affect aquatic diversity in the ICM. Generally, it appears that the 10% IC threshold applies to streams in the East Coast and Midwest, with Pacific Northwest streams showing impacts at a slightly higher level. For streams in the arid and semi-arid Southwest, it is unclear what, if any, IC threshold exists given the naturally stressful conditions for these intermittent and ephemeral streams (Maxted, 1999). Southwestern streams are characterized by seasonal bursts of short but intense rainfall and tend to have aquatic communities that are trophically simple and relatively low in species richness (Poff and Ward, 1989).

Overall, the following conclusions can be drawn:

- IC is the most commonly used index to assess the impacts of watershed urbanization on aquatic insect and fish diversity. Percent urban land use is also a common index.

- The ICM may not be sensitive enough to predict biological diversity in watersheds with low IC. For example, below 10% watershed IC, other watershed variables such as riparian continuity, natural forest cover, cropland, ditching and acid rain may be better for predicting stream health.

- More research needs to be done to determine the maximum level of watershed development at which stream diversity can be restored or maintained. Additionally, the capacity of stormwater treatment practices and stream buffers to mitigate high levels of watershed IC warrants more systematic research.

- More research is needed to test the ICM on amphibian and freshwater mussel diversity.

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Glossary

1st order stream: The smallest perennial stream. A stream that carries water throughout the year and does not have permanently flowing tributaries.

2nd order stream: Stream formed by the confluence of two 1st order streams.

3rd order stream: Stream formed by the confluence of two 2nd order streams.

Acute toxicity: Designates exposure to a dangerous substance or chemical with sufficient dosage to precipitate a severe reaction, such as death.

Alluvial: Pertaining to processes or materials associated with transportation or deposition by running water.

Anadromous: Organisms that spawn in freshwater streams but live most of their lives in the ocean.

Annual Pollutant Load: The total mass of a pollutant delivered to a receiving water body in a year.

Bankfull: The condition where streamflow just fills a stream channel up to the top of the bank and at a point where the water begins to overflow onto a floodplain.

Baseflow: Stream discharge derived from ground water that supports flow in dry weather.

Bedload: Material that moves along the stream bottom surface, as opposed to suspended particles.

Benthic Community: Community of organisms living in or on bottom substrates in aquatic habitats, such as streams.

Biological Indicators: A living organism that denotes the presence of a specific environmental condition.

Biological Oxygen Demand (BOD): An indirect measure of the concentration of biologically degradable material present in organic wastes. It usually reflects the amount of oxygen consumed in five days by bacterial processes breaking down organic waste.

Carcinogen: A cancer-causing substance or agent.

Catchment: The smallest watershed management unit. Defined as the area of a development site to its first intersection with a stream, usually as a pipe or open channel outfall.

Chemical Oxygen Demand (COD): A chemical measure of the amount of organic substances in water or wastewater. Non-biodegradable and slowly degrading compounds that are not detected by BOD are included.

Chronic Toxicity: Showing effects only over a long period of time.

Combined Sewer Overflow (CSO): Excess flow (combined wastewater and stormwater runoff) discharged to a receiving water body from a combined sewer network when the capacity of the sewer network and/or treatment plant is exceeded, typically during storm events.

Combined Indices (C-IBI or CSPS): Combined indices that use both fish and aquatic insect metrics and a variety of specific habitat scores to classify streams.

Cryptosporidium parvum: A parasite often found in the intestines of livestock which contaminates water when animal feces interacts with a water source.

Deicer: A compound, such as ethylene glycol, used to melt or prevent the formation of ice.

Dissolved Metals: The amount of trace metals dissolved in water.

Dissolved Phosphorus: The amount of phosphorus dissolved in water.

Diversity: A numerical expression of the evenness and distribution of organisms.

Ecoregion: A continuous geographic area over which the climate is uniform to permit the development of similar ecosystems on sites with similar geophysical properties.

Embeddedness: Packing of pebbles or cobbles with fine-grained silts and clays.

EPT Index: A count of the number of families of each of the three generally pollution-sensitive orders: Ephemeroptera (mayflies), Plecoptera (stoneflies), and Trichoptera (caddisflies).

Escherichia coli (E. coli): A bacteria that inhabits the intestinal tract of humans and other warm-blooded animals. Although it poses no threat to human health, its presence in drinking water does indicate the presence of other, more dangerous bacteria.

Eutrophication: The process of over-enrichment of water bodies by nutrients, often typified by the presence of algal blooms.

Fecal coliform: Applied to E. coli and similar bacteria that are found in the intestinal tract of humans and animals. Coliform bacteria are commonly used as indicators of the presence of pathogenic organisms. Their presence in water indicates fecal pollution and potential contamination by pathogens.

Fecal streptococci: Bacteria found in the intestine of warm-blooded animals. Their presence in water is considered to verify fecal pollution.

Fish Blockages: Infrastructures associated with urbanization, such as bridges, dams, and culverts, that affect the ability of fish to move freely upstream and downstream in watersheds. Can prevent re-colonization of resident fish and block the migration of anadromous fish.

Flashiness: Percent of flows exceeding the mean flow for the year. A flashy hydrograph would have larger, shorter-duration hydrograph peaks.

Geomorphic: The general characteristic of a land surface and the changes that take place in the evolution of land forms.

Giardia lamblia: A flagellate protozoan that causes severe gastrointestinal illness when it contaminates drinking water.

Herbicide: Chemicals developed to control or eradicate plants.

Hotspot: Area where land use or activities generate highly contaminated runoff, with concentrations of pollutants in excess of those typically found in stormwater.

Hydrograph: A graph showing variation in stage (depth) or discharge of a stream of water over a period of time.

Illicit discharge: Any discharge to a municipal separate storm sewer system that is not composed entirely of storm water, except for discharges allowed under an NPDES permit.

Impervious Cover: Any surface in the urban landscape that cannot effectively absorb or infiltrate rainfall.

Impervious Cover Model (ICM): A general watershed planning model that uses percent watershed impervious cover to predict various stream quality indicators. It predicts expected stream quality declines when watershed IC exceeds 10% and severe degradation beyond 25% IC.

Incision: Stream down-cuts and the channel expands in the vertical direction.

Index of Biological Integrity (IBI): Tool for assessing the effects of runoff on the quality of the aquatic ecosystem by comparing the condition of multiple groups of organisms or taxa against the levels expected in a healthy stream.

Infiltration: The downward movement of water from the surface to the subsoil. The infiltration capacity is expressed in terms of inches per hour.

Insecticide: Chemicals developed to control or eradicate insects.

Large Woody Debris (LWD): Fundamental to stream habitat structure. Can form dams and pools; trap sediment and detritus; provide stabilization to stream channels; dissipate flow energy and promote habitat complexity.

Mannings N: A commonly used roughness coefficient; actor in velocity and discharge formulas representing the effect of channel roughness on energy losses in flowing water.

Methyl Tertiary-Butyl Ether: An oxygenate and gasoline additive used to improve the efficiency of combustion engines in order to enhance air quality and meet air pollution standards. MTBE has been found to mix and move more easily in water than many other fuel components, thereby making it harder to control, particularly once it has entered surface or ground waters.

Microbe: Short for microorganism. Small organisms that can be seen only with the aid of a microscope. Most frequently used to refer to bacteria. Microbes are important in the degradation and decomposition of organic materials.

Nitrate: A chemical compound having the formula NO_3. Excess nitrate in surface waters can lead to excessive growth of aquatic plants.

Organic Matter: Plant and animal residues, or substances made by living organisms. All are based upon carbon compounds.

Organic Nitrogen: Nitrogen that is bound to carbon-containing compounds. This form of nitrogen must be subjected to mineralization or decomposition before it can be used by the plant community.

Overbank Flow: Water flow over the top of the bankfull channel and onto the floodplain.

Oxygenate: To treat, combine, or infuse with oxygen.

Peak Discharge: The maximum instantaneous rate of flow during a storm, usually in reference to a specific design storm event.

Pesticides: Any chemical agent used to control specific organisms, for example, insecticides, herbicides, fungicides and rodenticides.

Piedmont: Any plain, zone or feature located at the foot of a mountain. In the United States, the Piedmont (region) is a plateau extending from New Jersey to Alabama and lying east of the Appalachian Mountains.

Pool: A stream feature where there is a region of deeper, slow-moving water with fine bottom materials. Pools are the slowest and least turbulent of the riffle/run/pool category.

Protozoan: Any of a group of single-celled organisms.

Rapid Bioassessment Protocols (RBP): An integrated assessment, comparing habitat, water quality and biological measures with empirically defined reference conditions.

Receiving Waters: Rivers, lakes, oceans, or other bodies of water that receive water from another source.

Riffle: Shallow rocky banks in streams where water flows over and around rocks disturbing the water surface; often associated with whitewater. Riffles often support diverse biological communities due to their habitat niches and increased oxygen levels created by the water disturbance. Riffles are the most swift and turbulent in the riffle/run/pool category.

Roughness: A measurement of the resistance that streambed materials, vegetation, and other physical components contribute to the flow of water in the stream channel and floodplain. It is commonly measured as the Manning's roughness coefficient (Manning's N).

Run: Stream feature characterized by water flow that is moderately swift flow, yet not particularly turbulent. Runs are considered intermediate in the riffle/run/pool category.

Runoff Coefficient: A value derived from a site impervious cover value that is applied to a given rainfall volume to yield a corresponding runoff volume.

Salmonid: Belonging to the family Salmonidae, which includes trout and salmon.

Sanitary Sewer Overflow (SSO): Excess flow of wastewater (sewage) discharged to a receiving water body when the capacity of the sewer network and/or treatment plant is exceeded, typically during storm events.

Semi-arid: Characterized by a small amount of annual precipitation, generally between 10 and 20 inches.

Simple Method: Technique used to estimate pollutant loads based on the amount of IC found in a catchment or subwatershed.

Sinuosity: A measure of channel curvature, usually quantified as the ratio of the length of the channel to the length of a straight line along the valley axis. It is, in essence, a ratio of the stream's actual running length to its down-gradient length.

Soluble Phosphorus: The amount of phosphorus available for uptake by plants and animals.

Stormwater: The water produced as a result of a storm.

Subwatershed: A smaller geographic section of a larger watershed unit with a drainage area of between two to 15 square miles and whose boundaries include all the land area draining to a point where two 2nd order streams combine to form a 3rd order stream.

Total Dissolved Solids (TDS): A measure of the amount of material dissolved in water (mostly inorganic salts).

Total Kjeldhal Nitrogen (TKN): The total concentration of nitrogen in a sample present as ammonia or bound in organic compounds.

Total Recoverable Metals: The amount of a metal that is in solution after a representative suspended sediment sample has been digested by a method (usually using a dilute acid solution) that results in dissolution of only readily soluble substances).

Total Maximum Daily Load (TMDL): The maximum quantity of a particular water pollutant that can be discharged into a body of water without violating a water quality standard.

Total Nitrogen (Total N): A measure of the total amount of nitrate, nitrite and ammonia concentrations in a body of water.

Total Organic Carbon (TOC): A measure of the amount of organic material suspended or dissolved in water.

Total Phosphorous (Total P): A measure of the concentration of phosphorus contained in a body of water.

Total Suspended Solids (TSS): The total amount of particulate matter suspended in the water column.

Trophic Level: The position of an organism in a food chain or food pyramid.

Turbidity: A measure of the reduced transparency of water due to suspended material which carries water quality and aesthetic implications. Applied to waters containing suspended matter that interferes with the passage of light through the water or in which visual depth is restricted.

Volatile Organic Compounds (VOC): Chemical compounds which are easily transported into air and water. Most are industrial chemicals and solvents. Due to their low water solubility they are commonly found in soil and water.

The Association Between Extreme Precipitation and Waterborne Disease Outbreaks in the United States, 1948–1994

Frank C. Curriero, PhD, Jonathan A. Patz, MD, MPH, Joan B. Rose, PhD, and Subhash Lele, PhD

Objectives. Rainfall and runoff have been implicated in site-specific waterborne disease outbreaks. Because upward trends in heavy precipitation in the United States are projected to increase with climate change, this study sought to quantify the relationship between precipitation and disease outbreaks.

Methods. The US Environmental Protection Agency waterborne disease database, totaling 548 reported outbreaks from 1948 through 1994, and precipitation data of the National Climatic Data Center were used to analyze the relationship between precipitation and waterborne diseases. Analyses were at the watershed level, stratified by groundwater and surface water contamination and controlled for effects due to season and hydrologic region. A Monte Carlo version of the Fisher exact test was used to test for statistical significance.

Results. Fifty-one percent of waterborne disease outbreaks were preceded by precipitation events above the 90th percentile ($P = .002$), and 68% by events above the 80th percentile ($P = .001$). Outbreaks due to surface water contamination showed the strongest association with extreme precipitation during the month of the outbreak; a 2-month lag applied to groundwater contamination events.

Conclusions. The statistically significant association found between rainfall and disease in the United States is important for water managers, public health officials, and risk assessors of future climate change. (*Am J Public Health.* 2001;91:1194–1199)

According to the US National Assessment on the Potential Consequences of Climate Variability and Change,[1] determining the role of weather in the incidence of waterborne disease outbreaks is a priority public health research issue for this country. Rainfall and runoff have been implicated in individual outbreaks in the United Kingdom and the United States. A waterborne disease outbreak of giardiasis in Montana was related to rainfall,[2] as was the largest reported waterborne disease outbreak ever documented, which occurred in Milwaukee, Wis, in 1993. There, an estimated 403 000 cases of intestinal illness and 54 deaths occurred,[3] and the outbreak was preceded by a period of heavy rainfall and runoff with a subsequent turbidity load that compromised the efficiency of the drinking water treatment plant.[4,5]

Even outbreaks of *Escherichia coli,* generally considered a foodborne pathogen, have been linked to rainfall events. In fact, the largest reported outbreak of *E coli* O157:H7 occurred at a fairground in the state of New York in September 1999 and was linked to contaminated well water. Unusually heavy rainfall, which was preceded by a drought, coincided with this major outbreak.[1] Under conditions of high soil saturation, rapid transport of microbial organisms can be enhanced.

Part of the rationale for this study, conducted through a US Environmental Protection Agency grant for studying the effects of global climate change on public health, comes from projections of more intense rainfall that may accompany global warming. In the past century, average daily temperatures in the conterminous United States increased by approximately 1°F.[6] Warmer air can hold more moisture, and changes in the hydrologic cycle in the United States have been evidenced by increases in cloud cover[7] and total precipitation.[8] Moreover, the type of precipitation has been changing in the United States, with increases in extreme precipitation events (those with an intensity of more than 2 inches per day).[9,6,10] These rainfall patterns are consistent with expectations of a more vigorous hydrologic cycle caused by anthropogenic greenhouse gas warming of the earth's surface.[11–13]

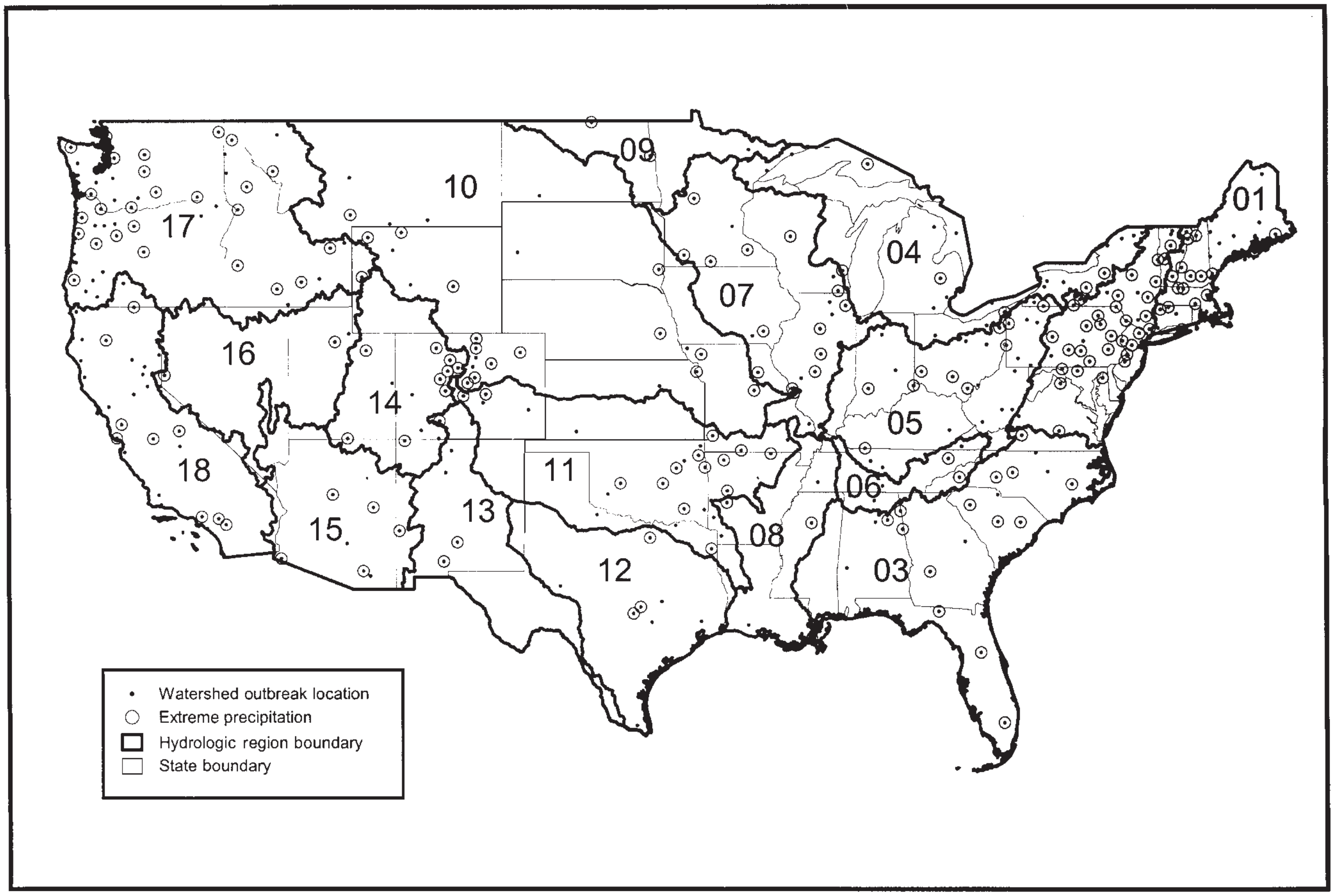
The purpose of our study was to analyze the relationship between precipitation and waterborne diseases, using the complete database of all reported waterborne disease outbreaks in the United States from 1948 to 1994. Rainfall intensity is assumed to be a key determining factor in the fate and transport of pathogenic microorganisms, but the relationship has never been analyzed at the national level.

METHODS

US Waterborne Disease Outbreaks and Precipitation Data Sets

Data on all reported waterborne disease outbreaks in the United States between 1948 and 1994 were obtained from the US Environmental Protection Agency's Office of Research and Development. Included in this data set were the etiologic agent, the community and state where the outbreak occurred, and the month and year of each outbreak. The outbreak source was designated as either surface water or groundwater contamination. The community and state information was geocoded and expressed as longitude and latitude coordinates marking the affected city or county.

A waterborne disease outbreak is defined as an outbreak in which epidemiologic evidence points to a drinking water source from which 2 or more persons become ill at similar times. All recreational outbreaks and outbreaks associated with cross-connections or back-siphonage between sewage and drinking water in the distribution system, including chemical outbreaks, were removed from the database. We excluded these outbreaks to focus the analysis on source waters and watershed contamination and to exclude acci-



Note. Outbreak locations represent the centroid of the affected watershed.

FIGURE 1—Waterborne disease outbreaks and associated extreme levels of precipitation (precipitation in the highest 10% [90th percentile]) within a 2-month lag preceding the outbreak month: United States, 1948–1994.

dental fecal releases associated with recreational outbreaks and infrastructure problems in the distribution system.

The conterminous United States is subdivided into 2105 hydrologic cataloging units called *watersheds*, which are geographic areas representing part or all of a surface drainage basin, a combination of drainage basins, or a distinct hydrologic feature. Watersheds act as the drinking water source for the surrounding area; thus, we chose watersheds as the geographic units for our investigation. Outbreak locations, originally designating the affected city or county, were recoded to correspond to the centroid of the associated watershed. Data on US hydrologic units, a hierarchy of geographic subdivisions including watersheds, were downloaded from the US Geological Survey.[14] Figure 1 includes boundaries for the largest subdivision in this hierarchy (watersheds are the smallest), which divides the United States into 18 distinct hydrologic regions, each containing the drainage area of a major river or the combined drainage areas of a series of rivers.

Total monthly precipitation readings for the more than 16 000 weather stations located across the United States from 1948 through 1994 were downloaded from the National Climatic Data Center.[15] The weather station locations were also coded to the watershed level; each watershed, on the average, contained approximately 7 weather stations. To account for local variations, we replaced recorded total monthly precipitation for each weather station with its corresponding *z* score, which was computed on the basis of the distribution of values recorded for that month from 1948 to 1997. We considered there to be sufficient information to compute *z* scores only if the corresponding distributions contained at least 20 years of recorded data. The *z* score thresholds were chosen to indicate extreme levels of precipitation. For example, *z* scores greater than 0.84, 1.28, and 1.65 correspond, respectively, to total monthly precipitation in the highest 20%, 10%, and 5% observed for that station and month from 1948 to 1994. The maximum *z* score determined from weather station–specific *z* scores within a watershed was used as a measure of extreme precipitation for that watershed.

Statistical Analysis

Figure 1 displays the 548 waterborne disease outbreaks, plotted using the centroid of the affected watershed, within the contermi-

TABLE 1—Waterborne Disease Outbreaks, With Associated Extreme Levels of Precipitation[a] in the Preceding 2 Months: United States, 1948–1994

	Extreme Precipitation		
Outbreak	Yes	No	Total
Yes	268	257	525
No	NC	NC	1 186 695
Total	NC	NC	1 187 220

Note. There were 1 187 220 watershed outbreak possibilities. Shown are the 525 outbreaks for which extreme precipitation data were available. Information regarding extreme precipitation status for watersheds not experiencing an outbreak was not compiled (NC).
[a]Precipitation in the highest 10% (90th percentile).

nous United States that were reported from 1948 to 1994. Of these outbreaks, 51% were preceded within a 2-month lag by an extreme level of precipitation in the highest 10% (or 90th percentile), as indicated in the figure. Several methods, and an accompanying large body of literature, are available to test for spatial clustering of disease events.[16] In this study we were interested in testing whether the outbreaks cluster around extreme precipitation events, as opposed to solely investigating geographic clustering of outbreaks.

Information in Figure 1 can be represented with a 2×2 contingency table, watershed outbreak status×watershed extreme precipitation status. Since this information is collapsed over time, there are a total of 1 187 220 watershed outbreak possibilities (47 years×12 months×2105 watersheds). Table 1 displays extreme precipitation status for only those watersheds known to have experienced an outbreak. Enumerating the bottom row would require determining the extreme precipitation status within a 2-month lag for the remaining watershed outbreak possibilities, a computational burden we wished to avoid. The total number of outbreaks is shown to be 525, not 548, because sufficient precipitation data were not available for 23 outbreak-associated watersheds.

Associations between events in contingency tables are usually described with odds ratios followed by a χ^2-based test of independence. Proceeding in this fashion, however, would require a completely enumerated table. Note that the percentage of coincident events reported (51%) is simply the (1,1) cell (outbreak and extreme precipitation) divided by its marginal total (number of outbreaks). Since the row and column totals in Table 1 are fixed, the (1,1) cell determines the remaining cells and hence the odds ratio; thus, the percentage of coincident events and the odds ratio are equivalent descriptors of association. Also, because the marginal totals are fixed, the Fisher exact test[17] can be used to assess the significance of the association based on the percentage of coincident events. Although the calculation of *P* values in the Fisher exact test requires fully enumerated information as well, the rationale behind the calculation can be approximated with the following Monte Carlo simulation.

The general idea is to repeatedly generate sets of "outbreaks" in a random fashion, tabulating the percentage of these artificial outbreaks that coincide with extreme levels of precipitation at each step. Such a process would produce a distribution of coincident percentages under the assumption of no association, which can then be compared with the observed percentage to compute a *P* value. The following algorithm describes the process for a given set of outbreaks overlaid with extreme precipitation events.

1. Generate a set of outbreaks.
   a. Randomly select watersheds.
   b. Randomly select a month (1–12) and year (1948–1994) for each watershed.
2. Calculate and store the percentage of these outbreaks coincident with extreme levels of precipitation within a given preceding monthly lag.
3. Repeat steps 1 and 2 one thousand times.

The expected percentage of outbreaks coincident with extreme levels of precipitation within a given preceding monthly lag, under the assumption of no association, can be estimated by averaging the Monte Carlo distribution of percentages in step 2.

For the data shown in Table 1, if the 525 waterborne disease outbreaks are clustered both spatially and temporally within watersheds experiencing extreme levels of precipitation, then the observed 51% would be higher than the percentage expected under the assumption of no association. We were therefore interested in testing the one-sided alternative representing a positive association between outbreaks and extreme precipitation. *P* values for such a test can be obtained by dividing by 1000 the number of percentages in step 2 that are higher than their respective observed percentages.

RESULTS

Table 2 cross-tabulates the 548 reported waterborne disease outbreaks by the 18 hydrologic regions and 4 seasons. The distribution of outbreaks across the seasons (column totals) shows that the number of outbreaks is highest during the summer months and lowest during the winter months. The distribution across the hydrologic regions (row totals) may be due to specific hydrologic features present in these regions. The distributional variations across regions and seasons can be controlled for in the Monte Carlo test by restricting the randomization scheme in step 1 of that algorithm to adhere to the marginal totals shown in Table 2. Thus, each artificial set of outbreaks would have identical row and column totals, as shown in Table 2. The resulting test would then be one of conditional association between outbreaks and extreme precipitation, controlling for variations across both regions and seasons.

Of the 548 waterborne disease outbreaks reported between 1948 and 1994, 133 (approximately 24%) were known to be from surface water contamination, 197 (approximately 36%) were known to be from groundwater contamination, and 218 (approximately 40%) had an unknown water contamination source. The outbreak data also included the etiologic agents involved in each outbreak. More than

TABLE 2—Waterborne Disease Outbreaks, by Hydrologic Region and Season: United States, 1948–1994

	Season				
Region	Winter	Spring	Summer	Fall	Total
1	2	8	17	11	38
2	14	27	63	29	133
3	4	5	12	8	29
4	6	2	18	8	34
5	6	9	18	6	39
6	1	1	2	3	7
7	2	12	10	3	27
8	1	1	5	2	9
9	1	0	1	1	3
10	5	5	24	7	41
11	6	9	16	8	39
12	0	3	4	2	9
13	0	1	5	1	7
14	6	6	7	4	23
15	1	3	3	1	8
16	0	1	3	0	4
17	6	17	34	8	65
18	9	6	14	4	33
Total	70	116	256	106	548

Note. Winter = December, January, February; Spring = March, April, May; Summer = June, July, August; Fall = September, October, November.

half the outbreaks were determined to be "acute gastrointestinal illness," about 13% were attributed to *Giardia,* and the remainder were caused by 35 other specific agents.

We used the Monte Carlo test presented above to test the significance of the overlaid information shown in Figure 1 and other associations between waterborne disease outbreaks and extreme precipitation, controlling for the possible confounding effects due to hydrologic region and season. Different scenarios were investigated by varying the preceding monthly lag time and level of extreme precipitation. Separate analyses were performed for outbreaks due to surface water contamination, outbreaks due to groundwater contamination, and the combined data, including outbreaks with an unknown water contamination source. The results, which are presented in Table 3, include for each scenario the observed percentage of outbreaks coincident with extreme precipitation events; an estimated expected percentage of coincident events, assuming no association; and the *P* value testing the significance of the observed percentage.

Results for the association depicted in Figure 1 (combined data, monthly lag 0, 1, 2, and 90th percentile extreme precipitation) indicate that after controlling for variations across regions and seasons, we would have expected 43.2% of the outbreaks to be coincident with extreme precipitation if there was no association between outbreaks and extreme precipitation. The observed percentage of outbreaks coincident with levels of extreme precipitation—51.0%—was highly significant (P=.002). *P* values of less than .001 in Table 3 indicate the strongest evidence of an association; they occurred when the random selection of watershed outbreaks, for the 1000 iterations performed in step 1 of the Monte Carlo algorithm, did not produce a percentage of outbreaks coincident with this level of extreme precipitation that was higher than the observed percentage.

The association between outbreaks and extreme precipitation remained statistically significant at the .05 level across all of the scenarios we considered for the combined data. The analysis stratified by water contamination source showed that outbreaks due to surface water contamination were most significant for extreme precipitation during the month of the outbreak. Outbreaks due to groundwater contamination, however, showed highest significance for extreme precipitation 2 months prior to the outbreak. This might be expected, considering the direct vs complex routes of exposure.

DISCUSSION

This study represents the first quantitative analysis of the relationship between extreme precipitation and waterborne disease outbreaks at the national level and over an extended period. Our findings show a statistically significant association between weather events and disease. However, we recognize that multiple factors are involved, which must occur simultaneously in time and space. Elements of an outbreak event include (1) a source of contamination (infected humans, domestic animals, or wildlife); (2) fate and transport of the contaminant from source to drinking water supplies; (3) inadequate treatment; and (4) detection and reporting of the outbreak.[18] Given the variability of these factors across the United States, the robustness of our findings demonstrates the important role of extreme wet-weather events in microbial fate and transport and as a contributing factor in US waterborne disease outbreaks.

Incorporating data on other causal components will be important in the development of better predictive models extending beyond this study's limitations. We have partially controlled for source of outbreak by conducting analyses at the watershed level. Watersheds might be expected to maintain some consistency in land use patterns; however, these patterns, inevitably, have changed over the 47 years analyzed. Several state-specific analyses that could include more detailed land use and treatment facility information would, therefore, be of benefit as a follow-up to this national-level study.

Our study is limited by the temporal resolution of the waterborne disease outbreak

TABLE 3—Monte Carlo Simulation Results for the Association Between Waterborne Disease Outbreaks and Extreme Precipitation: United States, 1948–1994

	Extreme Precipitation Percentile								
	Surface Water Contamination			Groundwater Contamination			Combined		
Monthly Lag	80th	90th	95th	80th	90th	95th	80th	90th	95th
Monthly lag 0									
Observed, %	39.1	28.9	22.7	31.2	21.4	13.5	33.3	22.8	16.8
Monte Carlo, %	26.9	17.4	11.7	28.8	18.6	12.4	27.7	17.9	12.0
P	.001	<.001	.001	.229	.173	.314	.001	<.001	.002
Monthly lag 0,1									
Observed, %	55.1	41.7	33.9	53.9	39.3	26.2	52.3	38.3	28.8
Monte Carlo, %	45.5	31.2	21.7	48.0	33.0	22.7	46.5	31.9	22.0
P	.022	.003	.002	.059	.039	.132	.003	.001	<.001
Monthly lag 0,1,2									
Observed, %	65.9	50.8	42.9	71.6	52.1	36.8	68.0	51.0	39.4
Monte Carlo, %	58.9	42.3	30.3	61.6	44.4	31.6	59.9	43.2	30.7
P	.063	.023	.001	.002	.021	.062	<.001	.002	<.001
Monthly lag 1									
Observed, %	34.6	22.8	18.1	33.2	22.8	14.5	31.6	20.3	14.9
Monte Carlo, %	26.8	17.4	11.6	28.7	18.5	12.3	27.5	17.7	11.8
P	.033	.060	.026	.083	.070	.183	.005	.047	.009
Monthly lag 1,2									
Observed, %	54.8	36.5	31.0	57.8	41.7	28.6	54.4	37.5	27.8
Monte Carlo, %	45.4	31.0	21.5	47.7	32.6	22.4	46.3	31.6	21.7
P	.023	.109	.003	.002	.009	.027	<.001	.001	<.001

Note. Shown are results for outbreaks known to be from surface water contamination, outbreaks known to be from groundwater contamination, and the combined data, including outbreaks with an unknown water contamination source. Listed for each monthly lag and extreme precipitation scenario are the observed percentage of outbreaks coincident with extreme precipitation, the Monte Carlo–expected percentage of coincident events, and the corresponding *P* value.

data. These data have been reported in the same way for approximately 50 years. Improved understanding and better prevention might be achieved if outbreak data included start and end dates rather than simply the month of occurrence.[18]

Reporting bias is a key component in the waterborne disease outbreak data. Experts estimate that we may be seeing only a small fraction of the actual outbreaks.[19] With such a bias, many of the cluster detection methods that focus primarily on geographic clustering of diseases would clearly be inappropriate. The method we applied, which is focused more on the clustering of outbreaks around extreme precipitation, is appropriate under the assumption that outbreak reporting is independent of surrounding monthly precipitation.

Although the United States is thought to have high-quality drinking water, the risk of contamination from leaking septic tanks or agricultural runoff remains. One pathogen, *Cryptosporidium*, a protozoan that completes its life cycle within the intestine of mammals, is shed in high numbers of infectious oocysts that are dispersed in feces. It is highly prevalent in ruminants and readily transmitted to humans.[20] In a cross-sectional analysis of 50 livestock farms sampled within the 100-year floodplain in Lancaster County, Pennsylvania, manure samples from 64% of the farms tested positive for *C parvum*.[21] Therefore, it is biologically plausible that increases in rainfall and runoff intensity would result in more contamination of source waters by this parasite.

Our results are also consistent with findings from other studies. For example, Atherholt et al. found that concentrations of *Cryptosporidium* oocysts and *Giardia* cysts in the Delaware River were positively correlated with rainfall.[22] In 1998, a drinking water outbreak of cryptosporidiosis that occurred in Brushy Creek, Tex, was linked to storms that led to sewage contamination of wells and creeks.[23] *Cryptosporidium* oocysts are very small (~5 microns) and are difficult to remove from water; a recent study found that 13% of finished water still contained *Cryptosporidium* oocysts,[24] indicating some passage of microorganisms from source to treated drinking water.

Municipal water systems, even today, can be overburdened by extreme rainfall events. For example, many communities still have combined sewer systems designed to carry both storm water and sanitary wastewater to a sewage treatment plant. During periods of heavy rainfall or snowmelt, the stormwater can exceed the capacity of the sewer system or treatment plant, and these systems are designed to discharge the excess wastewater directly into surface water bodies.[25,26] For northern latitudes and high-elevation regions, the addition of temperature values could further enhance the analysis by addressing the contribution of snowmelt.

During the heavy rainfall that accompanied the very strong El Niño of 1997 and 1998, a survey of a southwest Florida estuary found higher concentrations of fecal indicator organisms than occurred throughout the rest of the year,[27,28] implicating heavy rainfall as a risk factor for waterborne or seafood-borne disease. In urban watersheds, more than 60% of the annual load of all contaminants is transported during storm events.[29] In general, turbidity increases during storm events, and studies have recently shown a correlation between increases in turbidity and illness in communities.[30,31]

In summary, there is mounting evidence that heavy precipitation and runoff events significantly contribute to the risk of waterborne disease outbreaks. In the future, incorporation of other site-specific parameters, particularly land use patterns and treatment facility specifications, may allow for the development of more localized predictive models that can benefit water managers and public health planners. Our findings provide further insight into the linkage between weather and human disease that can be applied to risk assessments of future climate change. ■

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Contributors

F.C. Curriero developed the statistical methodology and performed all data analyses. J.A. Patz was the principal investigator for this study and conceived and led the overall design of this project. J.B. Rose obtained all the outbreak data and was responsible for plotting them, using a geographic information systems (GIS) format; provided information on the details of the database; and reviewed the article. S. Lele provided expert guidance on the statistical analyses and made revisions to the manuscript.

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The Health Effects of Swimming in Ocean Water Contaminated by Storm Drain Runoff

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Waters adjacent to the County of Los Angeles (CA) receive untreated runoff from a series of storm drains year round. Many other coastal areas face a similar situation. To our knowledge, there has not been a large-scale epidemiologic study of persons who swim in marine waters subject to such runoff. We report here results of a cohort study conducted to investigate this issue. Measures of exposure included distance from the storm drain, selected bacterial indicators (total and fecal coliforms, enterococci, and *Escherichia coli*), and a direct measure of enteric viruses. We found higher risks of a broad range of symptoms, including both upper respiratory and gastrointestinal, for subjects swimming (a) closer to storm drains, (b) in water with high levels of single bacterial indicators and a low ratio of total to fecal coliforms, and (c) in water where enteric viruses were detected. The strength and consistency of the associations we observed across various measures of exposure imply that there may be an increased risk of adverse health outcomes associated with swimming in ocean water that is contaminated with untreated urban runoff. (Epidemiology 1999;10:355–363)

Keywords: environmental epidemiology, gastrointestinal illness, ocean, recreational exposures, sewage, storm drains, waterborne illnesses, waterborne pathogens.

Runoff from a system of storm drains enters the Santa Monica Bay adjacent to Los Angeles County (CA). Even in the dry months of summer 10–25 million gallons of runoff (or non-storm water discharge) per day enter the bay from the storm drain system. Storm drain water is not subject to treatment and is discharged directly into the ocean. Total and fecal coliforms, as well as enterococci, are sometimes elevated in the surf zone adjacent to storm drain outlets; pathogenic human enteric viruses have also been isolated from storm drain effluents, even when levels of all commonly used indicators, including F2 male-specific bacteriophage, were low.[1]

Approximately 50–60 million persons visit Santa Monica Bay beaches annually. Concern about possible adverse health effects due to swimming in the bay has been raised by numerous interested parties.[2] Previous reports indicate that swimming in polluted water (for example, due to sewage) increases risks of numerous adverse health outcomes (Pruss[3] provides a recent review of this literature). To our knowledge, however, there has never been a large epidemiologic study of persons who swim in marine waters contaminated by heavy urban runoff.

These circumstances provided the motivation to study the possible health effects of swimming in the bay. We present here the main results from a large cohort study of people that addressed the issue of adverse health effects of swimming in ocean water subject to untreated urban runoff.

Methods

DESIGN AND SUBJECTS

The exposures of interest were distance swimming from storm drains, levels of bacterial indicators (total coli-

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forms, fecal coliforms, enterococcus, *Escherichia coli*) for pathogens that potentially produce acute illness, and human enteric viruses. We studied three beaches located in Santa Monica Bay (CA) that exhibited a wide range of pathogen indicator counts and a high density of swimmers (Santa Monica, Will Rogers, and Surfrider).

Persons who immersed their heads in the ocean water were potential subjects for this study. There was no restriction based on age, sex, or race. We excluded anyone who swam at the study beaches or in heavily polluted areas (that is, Mothers' Beach in Marina del Rey or near the Santa Monica Pier) within 7 days before the study date, or between the date of the beach interview and the telephone follow-up interview. We excluded subjects who swam on multiple days, as one of our primary questions was whether risk of health outcomes was associated with levels of indicator organisms on the specific day a subject entered the water. We targeted persons bathing within 100 yards upcoast or downcoast of the storm drain and persons bathing greater than 400 yards beyond a storm drain.

For this study, 22,085 subjects were interviewed on the beach from June 25 to September 14, 1995, to ascertain eligibility and willingness to participate. We found that 17,253 of these subjects were eligible and able to participate (that is, had a telephone and were able to speak English or Spanish). Of these, 15,492 (90% of the eligible subjects) agreed to participate. They were interviewed about their age, residence, and swimming, particularly immersion of the head into ocean water. The interviewer noted distance from the storm drain (within the categories 0, 1–50, 51–100, or 400 yards), gender, and race of the subject. (Distances from each drain were marked with inconspicuous objects such as beach towels and umbrellas.)

Nine to 14 days after the beach interview, subjects were interviewed by telephone to ascertain the occurrence(s) of: fever, chills, eye discharge, earache, ear discharge, skin rash, infected cuts, nausea, vomiting, diarrhea, diarrhea with blood, stomach pain, coughing, coughing with phlegm, nasal congestion, and sore throat. For this study we defined *a priori* three groupings of symptoms indicative of gastrointestinal illness or respiratory disease. In particular, following Cabelli *et al*,[4] subjects were classified as having highly credible gastrointestinal illness 1 (HCGI 1) if they experienced at least one of the following: (1) vomiting, (2) diarrhea and fever, or (3) stomach pain and fever. We also classified subjects as having highly credible gastrointestinal illness 2 (HCGI 2) if they had vomiting and fever. Finally, we classified subjects as having significant respiratory disease (SRD) if they had one of the following: (1) fever and nasal congestion, (2) fever and sore throat, or (3) coughing with phlegm.

We were able to contact and interview 13,278 subjects (86% follow-up). Of those interviewed, 1,485 were found to be ineligible because they swam (and immersed their heads) at a study beach or in heavily polluted waters between the day of the beach interview and the telephone follow-up. We excluded 107 subjects because they did not confirm immersing their faces in ocean water, leaving 11,686 subjects. One subject had a missing value for age, which we imputed (as the median value among all subjects) for inclusion in the adjusted analyses (discussed below). For the bacteriological analyses, we excluded an additional 1,227 subjects who had missing values, leaving 10,459 subjects. In the virus analyses we included only the 3,554 subjects who swam within 50 yards of the drain on days when viruses were measured (as the samples were collected only at the storm drain).

Collection and Analysis of Samples for Bacterial Indicators

Samples were collected on days that subjects were interviewed on the beaches. Each day, ankle depth samples were collected from each location (0 yards, 100 yards upcoast and downcoast of the drain, and one sample at 400 yards). One duplicate sample per site was collected daily. Samples were collected in sterile 1 liter polypropylene bottles and transferred on ice to the microbiology laboratory. All samples were analyzed for total coliforms, fecal coliforms, enterococcus, and *E. coli*. Densities of total and fecal coliforms and enterococci were determined using the appropriate membrane filtration techniques in Ref 5. *E. coli* densities were determined by membrane filtration using Hach Method 10029 for m-ColiBlue24 Broth.

Collection and Analysis of Samples for Enteric Viruses

For looking at enteric viruses, we collected samples from the three storm drain sites on Fridays, Saturdays, and Sundays, using Method 9510 C g of Ref 5. Ambient pH, temperature, conductivity, and total dissolved solids were measured. Samples as large as 100 gallons chosen to minimize the impacts of seawater dilution were filtered through electropositive filters at ambient pH. Adsorption filters were eluted in the field with 1 liter of sterile 3% beef extract adjusted to pH 9.0 with sodium hydroxide. Field eluates were reconcentrated in the laboratory using an organic reflocculation procedure.[6] All final concentrates were detoxified before analysis.[7]

All samples were analyzed for infectious human enteric viruses in Buffalo green monkey kidney cells (BGMK) by the plaque assay technique. Ten percent of the final concentrate was tested in this manner to determine whether there were a quantifiable number of viruses present. The remaining concentrate volume was divided in half and analyzed using the liquid overlay technique known as the cytopathic effect (CPE) assay.[8] The CPE assay generally detects a greater number of viruses than the plaque assay, but it is not quantitative. Flasks that did not exhibit CPE were considered to be negative for detectable infectious virus. We further examined any flask exhibiting CPE by the plaque-forming unit method to confirm the presence of infectious viruses.

Statistical Analysis

Our analysis addressed two main questions. First, are there different risks of specific outcomes among subjects swimming 0, 1–50, 51–100, and 400 or more yards from a storm drain? If pathogens in the storm drain result in increased acute illnesses, one would expect higher risks among swimmers closer to the drain. Second, are risks of specific outcomes associated with levels of specific bacterial indicators or enteric viruses?

To address the second question, we estimated risks arising from exposure to levels within categories defined *a priori* by existing standards or expert consensus. Specifically, for total coliforms we defined categories using 1,000 and 10,000 colony-forming units (cfu) per 100 ml as cutpoints, which are based on the California Code of Regulations (S.7958 in Title 17).[9] For fecal coliforms we created categories using cutpoints of 200 and 400 cfu per 100 ml, which reflect criteria set by the State Water Resources Control Board.[10] For enterococcus we used cutpoints of 35 and 104 cfu per 100 ml of water, which were established by the U.S. Environmental Protection Agency.[11] Finally, categories for *E. coli* were selected in meetings with staff from the Santa Monica Bay Restoration Project (SMBRP), Heal the Bay, and the Los Angeles County Department of Health Services. These meetings resulted in initially selecting categories based on cutpoints of 35 and 70 cfu per 100 ml, and then subsequently adding categories using cutpoints of 160 and 320 cfu per 100 ml; the latter were added because it is believed that *E. coli* comprises about 80% of the fecal coliforms. Using these knowledge-based categories, however, assumes a homogeneous risk between cutpoints. This might not be a reasonable assumption because the adequacy of these cutpoints is unclear, and because a large percentage of the subjects were in a single (that is, the lowest) category. Therefore, we further explored the bacteriological relations using categories defined by deciles.

In addition to considering total and fecal coliforms separately, we investigated the potential effect of the ratio of total to fecal coliforms. Motivation for this arose from our expectation that the risk of adverse health outcomes might be higher when the ratio is smaller, indicating a relatively greater proportion of fecal contamination. We used categories of this ratio defined by a cutpoint of 5 (where 5 corresponds to there being 5 times as much total as fecal coliform in the water). The human enteric virus exposure was reported as a dichotomous (that is, virus detected *vs* not detected) measure.

We first calculated simple descriptive statistics giving the number of subjects with each adverse health outcome who swam (1) at the prespecified distances from the drain or (2) in water with the prespecified levels of pathogens. From these counts we estimated the crude risk associated with each exposure. We then used logistic regression to estimate the adjusted relative risks of each outcome. For each exposure/outcome combination, we fit a separate model. All models adjusted for the potential confounding of: age (three categories: 0–12 years, 13–25 years, >25 years); sex; beach; race (four categories: white, black, Latino/a, and Asian/multiethnic/other); California *vs* out-of-state resident; and concern about potential health hazards at the beach (four categories: not at all, somewhat, a little, and very).

Results

Table 1 presents results for each of the adverse health outcomes by distance swimming from the storm drain. Across all distances, risks ranged from about 0.001 (that is, 1 per 1,000) for diarrhea with blood to about 0.1 for runny nose. The risk of numerous outcomes was higher for people who swam at the drain (0 yards away), in comparison with those who swam 1–50, 51–100, or >400 yards from the drain. In particular, we observed increases in risk for fever, chills, ear discharge, coughing with phlegm, HCGI 2, and SRD. In addition, the risks for eye discharge, earache, sore throat, infected cut, and HCGI 1 were also slightly elevated. A handful of outcomes exhibited small increased risks among swimmers at 1–50 yards (skin rash) or at 51–100 yards (cough, cough with phlegm, runny nose, and sore throat). Adjusted estimates of relative risk (RR) comparing swimmers at 0, 1–50, or 51–100 yards from the drain with swimmers at least 400 yards away from the drain showed similar relations as the aforementioned patterns of risks (Table 1). Among the positive associations for swimmers at the drain, RRs ranged in magnitude from about 1.2 (eye discharge, sore throat, HCGI 1) to 2.3 (earache), with varying degrees of precision; most of these RRs ranged from 1.4 to 1.6.

In Table 2 we see that the risk of skin rash increased for the highest prespecified category of total coliforms (that is, >10,000 cfu). Furthermore, the adjusted RR comparing swimmers exposed at this level *vs* those exposed to levels ≤1,000 cfu was 2.6. Whereas the RR for diarrhea with blood also suggested a positive association, this result was based on a single adverse health event (as evinced by the wide 95% CIs). When looking at deciles, in relation to the lowest exposure level (that is, the lowest 10%), we observed increased risks of skin rash at all other levels (Figure 1). The adjusted RRs ranged from 1.6 to 6.2, with five of the nine RRs in the 2–3 range. In addition, there were increased risks of HCGI 2 for all deciles except one (the eighth); the corresponding adjusted RRs ranged from 1.4 to 4.7, with varying levels of precision (Figure 1).

When looking at fecal coliforms, we again observed among those in the highest category (that is, >400 cfu) an increased risk for skin rash (Table 3). There were also *slight* increased risks for infected cut, runny nose, and diarrhea with blood in the highest category, as well as for nausea, vomiting, coughing, sore throat, and HCGI 2 in the middle category (200–400 cfu). The adjusted RRs also indicated positive associations for these outcomes (Table 3). When we used deciles to categorize subjects, however, in comparison with the lowest decile, we only observed marginal increased risks for infection and skin rash (not shown). In our investigation of the ratio of

TABLE 1. Adverse Health Outcomes by Distance Swimming from Drain: Number Ill, Acute Risks, Adjusted Relative Risk (RR) Estimates and 95% Confidence Intervals (CI)

	Distance from Drain (in Yards)										
	>400 (N = 3030)*		51–100 (N = 3311)			1–50 (N = 4518)			0 (N = 827)		
Outcome	No. Ill	Risk	No. Ill	Risk	RR (95% CI)†	No. Ill	Risk	RR (95% CI)†	No. Ill	Risk	RR (95% CI)†
Fever	138	0.046	158	0.048	1.06 (0.84–1.34)	208	0.046	1.07 (0.85–1.33)	59	0.071	1.61 (1.16–2.24)
Chills	72	0.024	85	0.026	1.07 (0.77–1.47)	108	0.024	1.05 (0.77–1.42)	31	0.037	1.60 (1.03–2.50)
Eye discharge	61	0.020	59	0.018	0.88 (0.61–1.27)	73	0.016	0.77 (0.55–1.09)	19	0.023	1.15 (0.67–1.98)
Earache	116	0.038	116	0.035	0.89 (0.68–1.16)	136	0.030	0.81 (0.63–1.04)	38	0.046	1.34 (0.91–1.98)
Ear discharge	21	0.007	19	0.006	0.78 (0.42–1.46)	25	0.006	0.80 (0.45–1.44)	13	0.016	2.09 (1.01–4.33)
Skin rash	23	0.008	30	0.009	1.16 (0.67–2.01)	53	0.012	1.50 (0.91–2.46)	4	0.005	0.62 (0.21–1.83)
Infected cut	17	0.006	16	0.005	0.79 (0.40–1.58)	37	0.008	1.51 (0.84–2.69)	6	0.007	1.48 (0.57–3.87)
Nausea	133	0.044	115	0.035	0.77 (0.60–1.00)	143	0.032	0.75 (0.59–0.95)	40	0.048	1.13 (0.78–1.65)
Vomiting	57	0.019	58	0.018	0.97 (0.67–1.40)	63	0.014	0.76 (0.53–1.09)	25	0.030	1.40 (0.85–2.31)
Diarrhea	204	0.067	163	0.049	0.70 (0.56–0.86)	202	0.045	0.69 (0.56–0.84)	53	0.064	1.04 (0.75–1.44)
Diarrhea with blood	7	0.002	2	0.001	0.26 (0.05–1.26)	3	0.001	0.27 (0.07–1.06)	2	0.002	0.87 (0.15–4.57)
Stomach pain	206	0.068	194	0.059	0.85 (0.70–1.05)	271	0.060	0.93 (0.77–1.12)	61	0.074	1.11 (0.82–1.51)
Cough	209	0.069	263	0.079	1.18 (0.97–1.42)	296	0.066	0.98 (0.82–1.18)	55	0.067	1.01 (0.73–1.38)
Cough and phlegm	90	0.030	114	0.034	1.16 (0.88–1.54)	143	0.032	1.09 (0.83–1.43)	39	0.047	1.65 (1.11–2.46)
Runny nose	273	0.090	351	0.106	1.18 (1.00–1.40)	371	0.082	0.95 (0.80–1.12)	74	0.089	1.10 (0.84–1.46)
Sore throat	190	0.063	244	0.074	1.17 (0.96–1.43)	304	0.067	1.12 (0.93–1.35)	59	0.071	1.25 (0.92–1.71)
HCGI 1	102	0.034	96	0.029	0.88 (0.66–1.17)	121	0.027	0.84 (0.64–1.10)	35	0.042	1.21 (0.81–1.82)
HCGI 2	26	0.009	28	0.008	1.04 (0.61–1.79)	32	0.007	0.90 (0.53–1.53)	15	0.018	1.64 (0.84–3.21)
Significant respiratory disease	139	0.046	177	0.053	1.18 (0.94–1.49)	205	0.045	1.03 (0.82–1.23)	63	0.076	1.78 (1.29–2.45)

The total number of swimmers in each category is given in parentheses (N). HCGI1, highly credible gastrointestinal illness with vomiting, diarrhea and fever or stomach pain and fever. HCGT2, highly credible gastrointestinal illness with vomiting and fever only. Significant respiratory disease, fever and nasal congestion, fever and sore throat or coughing with phlegm.
* Referent category (RR = 1.0).
† Adjusted for age, sex, beach, race, California *vs* out-of-state resident, and concern about potential health hazards at the beach.

total to fecal coliforms, we observed a consistent pattern of higher risks for diarrhea and HCGI 2 as the ratio category became lower (not shown, but available in Ref 12). Because any effect of this lower ratio should be stronger when there was a higher degree of contamination, indicated by total coliform counts in excess of 1,000 or 5,000 cfu, we then restricted our analysis to subjects swimming in water above these levels. In the first case, increased risks with decreasing cutpoints were observed for nausea, diarrhea, and HCGI 2.[12] When we restricted our investigation to subjects in water in which the total coliforms exceeded 5,000 cfu, we observed

TABLE 2. Adverse Health Outcomes by Total Coliform Levels: Number Ill, Acute Risks, Adjusted Relative Risk (RR) Estimates and 95% Confidence Intervals (CI)

	Total Coliforms (cfu/100ml)							
	≤1,000 (N = 7,574)*		>1,000–10,000 (N = 1,988)			>10,000 (N = 757)		
Outcome	No. Ill	Risk	No. Ill	Risk	RR†	No. Ill	Risk	RR†
Fever	368	0.049	88	0.044	0.92 (0.72–1.17)	42	0.055	1.23 (0.87–1.73)
Chills	193	0.025	51	0.026	1.03 (0.75–1.42)	9	0.012	0.51 (0.26–1.01)
Eye discharge	151	0.020	21	0.011	0.46 (0.29–0.74)	15	0.020	0.81 (0.47–1.41)
Earache	270	0.036	66	0.033	0.96 (0.72–1.27)	21	0.028	0.86 (0.54–1.38)
Ear discharge	51	0.007	15	0.008	1.22 (0.67–2.23)	2	0.003	0.46 (0.11–1.93)
Skin rash	65	0.009	14	0.007	0.75 (0.41–1.36)	19	0.025	2.59 (1.49–4.53)
Infected cut	49	0.006	11	0.006	0.97 (0.49–1.91)	3	0.004	0.82 (0.25–2.72)
Nausea	292	0.039	69	0.035	0.94 (0.72–1.24)	18	0.024	0.71 (0.43–1.16)
Vomiting	137	0.018	34	0.017	0.90 (0.61–1.33)	9	0.012	0.64 (0.32–1.29)
Diarrhea	434	0.057	85	0.043	0.80 (0.63–1.03)	33	0.044	0.95 (0.65–1.39)
Diarrhea with blood	8	0.001	2	0.001	1.08 (0.22–5.35)	1	0.001	1.73 (0.19–15.88)
Stomach pain	487	0.064	125	0.063	1.05 (0.85–1.29)	29	0.038	0.69 (0.47–1.02)
Cough	546	0.072	133	0.067	0.90 (0.73–1.10)	51	0.067	0.94 (0.69–1.28)
Cough and phlegm	267	0.035	58	0.029	0.81 (0.60–1.09)	27	0.036	1.03 (0.68–1.57)
Runny nose	703	0.093	170	0.086	0.93 (0.78–1.12)	67	0.089	1.06 (0.81–1.40)
Sore throat	534	0.071	116	0.058	0.83 (0.67–1.03)	47	0.062	0.95 (0.69–1.30)
HCGI 1	242	0.032	54	0.027	0.84 (0.62–1.14)	17	0.022	0.74 (0.44–1.23)
HCGI 2	72	0.010	16	0.008	0.89 (0.51–1.55)	5	0.007	0.83 (0.32–2.12)
Significant respiratory disease	396	0.052	84	0.042	0.80 (0.62–1.02)	42	0.055	1.11 (0.79–1.55)

The total number of swimmers in each category is given in parentheses (N).
* Referent category (RR = 1.0).
† Adjusted for age, sex, beach, race, California *vs* out-of-state resident, and concern about potential health hazards at the beach.

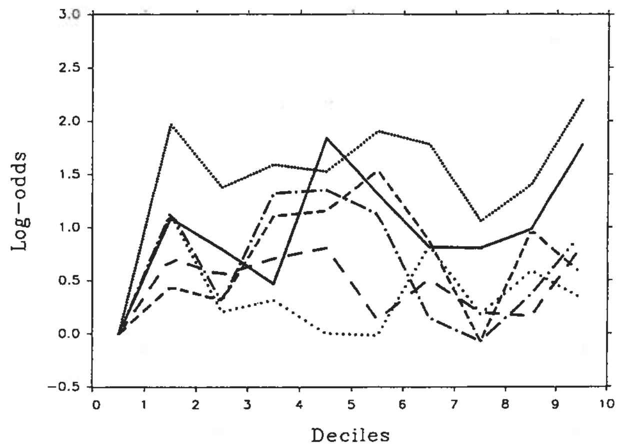

FIGURE 1. Log odds of adverse health outcomes by deciles of exposure for selected bacterial exposures. —, Total coliform and skin rash; – – – ·, total coliform and HCGI 2; · · ·, Enterococci and infected cut; — — —, *E coli* and eye discharge; ···, *E coli* and skin rash; · — ··, *E coli* and infected cut. HCGI 2 = highly credible gastrointestinal illness with vomiting and fever only.

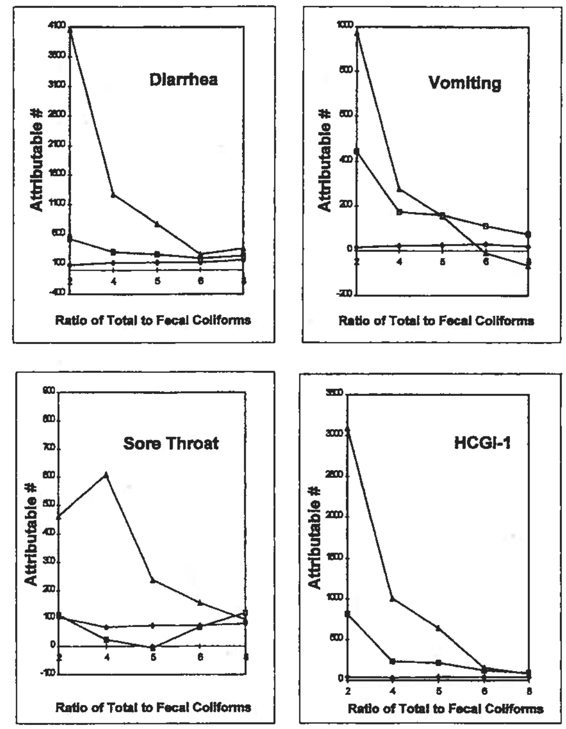

FIGURE 2. Selected attributable numbers/10,000 exposed subjects for total to fecal coliforms. ♦, All days; ■, >1000; ▲, > 5000. HCGI 1 = highly credible gastrointestinal illness with vomiting, diarrhea and fever or stomach pain and fever.

increased risks with eye discharge, ear discharge, skin rash, nausea, diarrhea, stomach pain, nasal congestion, HCGI 1, and HCGI 2.[12] There was a consistent pattern of stronger risk ratios as the cutpoint became lower (when the analyses were restricted to times when total coliforms exceeded 1,000 or 5,000 cfu), with the strongest effects generally observed with the cutpoint of 2, as illustrated in Figure 2 for diarrhea, vomiting, sore throat, and HCGI1.

Table 4 gives results for the relation among enterococci and the adverse health outcomes. Again, we observed an increased risk of skin rash among those in the highest category (that is, >104 cfu). In addition, comparing the highest to other categories of exposure, there

TABLE 3. Adverse Health Outcomes by Fecal Coliform Levels: Number Ill, Acute Risks, Adjusted Relative Risk (RR) Estimates and 95% Confidence Intervals (CI)

	Fecal Coliforms (cfu/100ml)							
	≤200 (N = 8,005)*		>200–400 (N = 768)			>400 (N = 1,636)		
Outcome	No. Ill	Risk	No. Ill	Risk	RR†	No. Ill	Risk	RR†
Fever	381	0.048	39	0.051	1.04 (0.74–1.46)	80	0.049	1.02 (0.80–1.32)
Chills	197	0.025	24	0.031	1.14 (0.74–1.76)	34	0.021	0.78 (0.54–1.14)
Eye discharge	149	0.019	11	0.014	0.70 (0.38–1.31)	30	0.018	0.97 (0.65–1.46)
Earache	275	0.034	26	0.04	0.93 (0.62–1.41)	57	0.035	1.00 (0.75–1.35)
Ear discharge	53	0.007	8	0.010	1.29 (0.60–2.73)	7	0.004	0.56 (0.25–1.24)
Skin rash	69	0.009	5	0.007	0.64 (0.26–1.60)	26	0.016	1.86 (1.17–2.95)
Infected cut	47	0.006	2	0.003	0.40 (0.10–1.65)	15	0.009	1.50 (0.83–2.74)
Nausea	289	0.036	38	0.049	1.29 (0.91–1.84)	57	0.035	0.93 (0.69–1.24)
Vomiting	133	0.017	18	0.023	1.33 (0.81–2.21)	31	0.019	1.07 (0.71–1.60)
Diarrhea	425	0.053	50	0.065	1.17 (0.86–1.60)	81	0.050	0.90 (0.70–1.15)
Diarrhea with blood	7	0.001	1	0.001	1.22 (0.15–10.01)	3	0.002	1.69 (0.42–6.75)
Stomach pain	495	0.062	51	0.066	1.04 (0.77–1.41)	103	0.063	0.98 (0.78–1.23)
Cough	551	0.069	70	0.091	1.34 (1.03–1.74)	117	0.072	1.06 (0.86–1.31)
Cough and phlegm	265	0.033	31	0.040	1.16 (0.79–1.70)	60	0.037	1.10 (0.82–1.47)
Runny nose	722	0.090	72	0.94	1.03 (0.79–1.33)	160	0.098	1.11 (0.93–1.34)
Sore throat	527	0.066	70	0.091	1.40 (1.07–1.82)	106	0.065	0.99 (0.80–1.24)
HCGI 1	239	0.030	28	0.036	1.18 (0.79–1.77)	50	0.031	0.99 (0.72–1.36)
HCGI 2	65	0.008	11	0.014	1.63 (0.85–3.12)	17	0.010	1.13 (0.65–1.95)
Significant respiratory disease	399	0.050	42	0.055	1.08 (0.77–1.50)	85	0.052	1.04 (0.81–1.33)

The total number of swimmers in each category is given in parentheses (N).
* Referent category (RR = 1.0).
† Adjusted for age, sex, beach, race, California *vs* out-of-state resident, and concern about potential health hazards at the beach.

TABLE 4. Adverse Health Outcomes by Enterococci Levels: Number Ill, Acute Risks, Adjusted Relative Risk (RR) Estimates and 95% Confidence Intervals (CI)

	Enterococci (cfu/100ml)							
	≤35 (N = 7,689)*		>35–104 (N = 1,863)			>104 (N = 857)		
Outcome	No. Ill	Risk	No. Ill	Risk	RR†	No. Ill	Risk	RR†
Fever	371	0.048	84	0.045	0.91 (0.71–1.16)	45	0.053	1.00 (0.72–1.40)
Chills	198	0.026	33	0.018	0.67 (0.46–0.97)	24	0.028	0.94 (0.60–1.48)
Eye discharge	149	0.019	25	0.013	0.69 (0.45–1.07)	16	0.019	1.01 (0.58–1.75)
Earache	270	0.035	57	0.031	0.82 (0.61–1.11)	31	0.036	0.88 (0.59–1.31)
Ear discharge	52	0.007	12	0.006	0.85 (0.45–1.62)	4	0.005	0.53 (0.19–1.51)
Skin rash	74	0.010	13	0.007	0.71 (0.39–1.30)	13	0.015	1.72 (0.89–3.31)
Infected cut	46	0.006	12	0.006	0.95 (0.49–1.82)	6	0.007	0.90 (0.37–2.18)
Nausea	271	0.035	72	0.039	1.07 (0.82–1.41)	41	0.048	1.19 (0.84–1.70)
Vomiting	130	0.017	34	0.018	1.13 (0.77–1.67)	18	0.021	1.20 (0.71–2.04)
Diarrhea	398	0.052	101	0.054	0.99 (0.78–1.25)	57	0.067	1.01 (0.75–1.36)
Diarrhea with blood	8	0.001	0	—	—	3	0.004	2.90 (0.66–12.68)
Stomach pain	464	0.060	126	0.068	1.09 (0.89–1.35)	59	0.069	0.97 (0.72–1.30)
Cough	554	0.072	121	0.065	0.91 (0.73–1.12)	63	0.074	1.00 (0.75–1.34)
Cough and phlegm	266	0.035	59	0.032	0.91 (0.68–1.22)	31	0.036	1.03 (0.69–1.54)
Runny nose	704	0.092	165	0.089	0.96 (0.80–1.15)	85	0.099	1.01 (0.79–1.30)
Sore throat	533	0.069	118	0.063	0.89 (0.72–1.10)	52	0.061	0.80 (0.59–1.09)
HCGI 1	230	0.030	51	0.027	0.92 (0.67–1.26)	36	0.042	1.31 (0.89–1.92)
HCGI 2	67	0.009	14	0.008	0.82 (0.46–1.48)	12	0.014	1.30 (0.67–2.51)
Significant respiratory disease	397	0.052	84	0.045	0.86 (0.67–1.11)	45	0.053	0.98 (0.70–1.37)

The total number of swimmers in each category is given in parentheses (N).
* Referent category (RR = 1.0).
† Adjusted for age, sex, beach, race, California *vs* out-of-state resident, and concern about potential health hazards at the beach.

were increased risks of nausea, vomiting, diarrhea with blood, HCGI 1, and HCGI 2. Our adjusted RRs suggested similar positive associations, except for diarrhea; although the risk increased from 0.05 to 0.07, the adjusted RR comparing the highest to lowest category was 1.0 (Table 4). When comparing the lowest to higher deciles, we observed increased risks in most categories for infected cut and skin rash (Figure 1). Other adverse health outcomes—infected cut, nausea, diarrhea, diarrhea with blood, HCGI 1, and HCGI 2—exhibited increased risks only in particular quantiles. In comparison with the lowest decile, the risk of each of these outcomes was higher in the 10th decile. For example, the risk for HCGI 2 was 0.007 in the first decile, but 0.015 in the 10th.

Table 5 presents results for *E. coli*. We once again found an increased risk of skin rash in the highest prespecified category (that is, >320 cfu). Furthermore, we observed slight increased risks in this highest category for eye discharge, earache, stomach pain, coughing with phlegm, runny nose, and HCGI 1 (Table 5). In our decile-based analysis, however, we only observed materially increased risks for eye discharge, skin rash, and infection (Figure 1).

Numerous adverse health outcomes exhibited higher risks among subjects swimming on days when samples were positive for viruses (Table 6). In particular, the risk of fever, eye discharge, vomiting, sore throat, HCGI 1, and HCGI 2, and to a lesser extent, chills, diarrhea, diarrhea with blood, cough, coughing with phlegm, and SRD were higher on days when viruses were detected. Our adjusted RR estimates showed similar relations, most ranging from 1.3 to 1.9 (Table 6). Additionally, adjusting for each bacterial indicator (one-at-a-time) also left these results essentially unchanged.[12] As expected, there was an association between presence of virus and fecal coliforms within 50 yards of the drain. The mean density of fecal coliforms when no virus was detected was 234.8 cfu (SD 542.5 cfu); whereas it was 2,233.8 (SD 2,634.1) when viruses were detected (N = 386). The median values were 47.8 and 452.6 cfu, respectively.

Discussion

We observed differences in risk for a number of outcomes when we compared subjects swimming at 0 yards *vs* 400+ yards. Most of the relative risks suggested an approximately 50% increase in risk. Furthermore, as evinced by both the risks and RRs, there is an apparent threshold of increased risk occurring primarily at the drain: no dose response is evinced with increasing closeness to the drain, but there is a jump in risk for many adverse health outcomes among those swimming at the drain. We also found that distance is a reasonably good surrogate for bacterial indicators, with higher levels observed closer to the drain.[12]

For bacterial indicators, we observed a relation among numerous higher exposures and adverse health outcomes. These increases were mostly restricted to the highest knowledge-based categories (no effect was observed below any existing standards). When looking at quantiles, we found higher risks of skin rash and infection at fairly low levels. In contrast with what one might expect, however, there was no clear dose-response pattern across increasing levels of bacteriological exposures.

TABLE 5. Adverse Health Outcomes by E. coli Levels: Number Ill, Acute Risks, Adjusted Relative Risk (RR) Estimates and 95% Confidence Intervals (CI)

	E. coli (cfu/100ml)													
	≤35 (N = 6,104)*		>35–75 (N = 1,620)			>75–160 (N = 1,145)			>160–320 (N = 518)			>320 (N = 991)		
Outcome	No. Ill	Risk	No. Ill	Risk	RR†	No. Ill	Risk	RR†	No. Ill	Risk	RR†	No. Ill	Risk	RR†
Fever	274	0.045	89	0.055	1.22 (0.95–1.56)	61	0.053	1.20 (0.90–1.60)	29	0.056	1.22 (0.81–1.84)	45	0.045	0.98 (0.70–1.37)
Chills	145	0.024	41	0.025	1.00 (0.70–1.44)	28	0.024	1.00 (0.66–1.52)	18	0.035	1.38 (0.82–2.33)	22	0.022	0.79 (0.49–1.26)
Eye discharge	116	0.019	30	0.019	0.99 (0.65–1.49)	14	0.012	0.65 (0.37–1.15)	6	0.012	0.61 (0.26–1.43)	23	0.023	1.36 (0.84–2.19)
Earache	214	0.035	45	0.028	0.75 (0.54–1.04)	33	0.029	0.78 (0.53–1.14)	18	0.035	0.91 (0.55–1.50)	47	0.047	1.25 (0.89–1.77)
Ear discharge	42	0.007	8	0.005	0.60 (0.28–1.28)	5	0.004	0.57 (0.22–1.46)	6	0.012	1.28 (0.52–3.15)	6	0.0066	0.67 (0.27–1.62)
Skin rash	57	0.009	15	0.009	1.01 (0.56–1.80)	7	0.006	0.66 (0.30–1.46)	6	0.012	1.21 (0.49–2.98)	15	0.015	2.04 (1.11–3.76)
Infected cut	42	0.007	7	0.004	0.53 (0.24–1.20)	3	0.003	0.33 (0.10–1.06)	3	0.006	0.66 (0.20–2.19)	9	0.009	1.02 (0.48–2.19)
Nausea	216	0.035	74	0.046	1.22 (0.93–1.61)	34	0.030	0.80 (0.55–1.16)	18	0.035	0.88 (0.53–1.46)	42	0.042	1.03 (0.73–1.47)
Vomiting	107	0.018	31	0.019	1.09 (0.72–1.64)	16	0.014	0.82 (0.48–1.40)	8	0.015	0.87 (0.41–1.85)	20	0.020	1.05 (0.63–1.74)
Diarrhea	310	0.051	101	0.062	1.14 (0.90–1.44)	63	0.055	1.00 (0.75–1.33)	25	0.048	0.80 (0.52–1.23)	56	0.057	0.91 (0.67–1.23)
Diarrhea with blood	5	0.001	3	0.002	2.06 (0.48–8.89)	1	0.001	1.03 (0.12–9.01)	2	0.004	3.98 (0.68–23.21)	0	—	—
Stomach pain	353	0.058	124	0.077	1.28 (1.03–1.59)	70	0.061	1.02 (0.78–1.33)	31	0.060	0.95 (0.64–1.40)	70	0.071	1.06 (0.80–1.40)
Cough	444	0.073	96	0.059	0.81 (0.64–1.02)	86	0.075	1.04 (0.82–1.33)	29	0.056	0.77 (0.51–1.14)	82	0.083	1.14 (0.88–1.48)
Cough and phlegm	226	0.037	41	0.025	0.66 (0.47–0.92)	34	0.030	0.78 (0.54–1.12)	11	0.021	0.53 (0.28–1.00)	43	0.043	1.12 (0.79–1.59)
Runny nose	566	0.093	136	0.084	0.87 (0.71–1.06)	105	0.092	0.96 (0.77–1.20)	38	0.073	0.76 (0.53–1.08)	108	0.109	1.12 (0.89–1.41)
Sore throat	417	0.068	99	0.061	0.86 (0.68–1.08)	82	0.072	1.02 (0.80–1.31)	29	0.056	0.78 (0.52–1.17)	75	0.076	1.04 (0.80–1.37)
HCGI 1	183	0.030	51	0.031	1.03 (0.75–1.42)	30	0.026	0.88 (0.59–1.30)	17	0.033	1.06 (0.63–1.80)	36	0.036	1.12 (0.76–1.64)
HCGI 2	48	0.008	21	0.013	1.55 (0.92–2.64)	8	0.007	0.85 (0.40–1.81)	6	0.012	1.25 (0.51–3.03)	10	0.010	1.04 (0.51–2.13)
Significant respiratory disease	319	0.052	71	0.044	0.82 (0.62–107)	58	0.051	0.96 (0.72–1.28)	21	0.041	0.74 (0.47–1.18)	56	0.057	1.03 (0.76–1.40)

The total number of swimmers in each category is given in parentheses (N).
* Referent category (RR = 1.0).
† Adjusted for age, sex, beach, race, California *vs* out-of-state resident, and concern about potential health hazards at the beach.

TABLE 6. Number Ill, Risks, and Adjusted Relative Risk (RR) Estimates of Adverse Health Outcomes by Virus

	Viruses				
	No (N = 3,168)*		Yes (N = 386)		
Outcome	No. Ill	Risk	No. Ill	Risk	RR (95% CI)†
Fever	126	0.040	23	0.060	1.56 (0.98–2.50)
Chills	65	0.021	10	0.026	1.25 (0.63–2.50)
Eye discharge	36	0.011	8	0.021	1.86 (0.85–4.09)
Earache	93	0.029	10	0.026	0.92 (0.47–1.80)
Ear discharge	15	0.005	0		
Skin rash	32	0.010	4	0.010	0.97 (0.34–2.82)
Infected cut	31	0.010	2	0.005	0.57 (0.13–2.40)
Nausea	101	0.032	12	0.031	0.93 (0.50–1.73)
Vomiting	44	0.014	10	0.026	1.86 (0.92–3.80)
Diarrhea	130	0.041	21	0.054	1.27 (0.78–2.07)
Diarrhea with blood	2	0.001	1	0.003	5.82 (0.45–75.72)
Stomach pain	191	0.060	23	0.060	0.92 (0.58–1.45)
Cough	181	0.057	28	0.073	1.22 (0.80–1.86)
Cough and phlegm	92	0.029	13	0.034	1.20 (0.66–2.18)
Runny nose	246	0.078	32	0.083	1.01 (0.68–1.49)
Sore throat	198	0.063	32	0.083	1.38 (0.93–2.06)
HCGI 1	72	0.023	15	0.039	1.69 (0.95–3.01)
HCGI 2	22	0.007	6	0.016	2.32 (0.91–5.88)
Significant respiratory disease	133	0.042	21	0.054	1.34 (0.83–2.18)

The total number of swimmers in each category is given in parentheses (N).
* Referent category (RR = 1.0).
† Adjusted for age, sex, beach, race, California *vs* out-of-state resident, and concern about potential health hazards at the beach.

When looking at the ratio of total to fecal coliforms using the entire dataset, no consistent pattern emerged.[12] This is not entirely surprising inasmuch as an analysis of all data points treats all ratios of similar numerical value equally. Thus, for example, even though a ratio of 5 when the total coliforms are very low may not increase risk, the same ratio may be associated with increased risks when the density of total coliforms is above 1,000 or 5,000 cfu. When the analysis was restricted to swimmers exposed to total coliform densities above 1,000 or 5,000 cfu, a consistent pattern emerged, with higher risks associated with low ratios.[12]

This is the first large-scale epidemiologic study that included measurements of viruses. A number of adverse health effects were reported more often on days when the samples were positive, suggesting assays for viruses may be informative for predicting risk. Norwalk-like viruses are a plausible cause of gastroenteritis.[4,13] Enteroviruses, the most common viruses in sewage effluent, can cause respiratory symptoms. Not only are viruses responsible for many of the symptoms associated with swimming in ocean water but also they die off at slower rates in sea water than do bacteria, and they can cause infection at a much lower dose.[14]

Our design substantially reduced the potential for confounding by restricting the study entirely to swimmers and making comparisons between groups of swimmers (for example, defined by distance from the drain) to estimate relative risks. Previous studies looking at the effects of exposure to polluted recreational water (for example, due to sewage outflows) have been criticized for comparing risks in swimmers with risks in nonswimmers.[4,14,15] In these earlier studies, background risks among subjects who swim *vs* those choosing not to swim may differ because there are many other (potentially noncontrollable) exposures/pathways that can produce the symptoms under investigation. By restricting the present study to swimmers, we have reduced potential differences between the background risks of exposed *vs* unexposed subjects (for example, swimmers choosing to swim at the drain *vs* those swimming at the same beach but farther away from the drain). Furthermore, we were able to adjust our relative risk estimates for a number of additional factors (listed above) that could confound the observed relations. Of course, this does not exclude the possibility that residual confounding in these factors, or other unknown factors, might have confounded the observed relations.

Nevertheless, any actual (that is, causal) effects may be higher than we observed in this study because both distance and pathogenic indicators are proxy measures of the true pathogenic agents. Also, recall that we excluded subjects who frequently entered the water at these beaches. If there is a dose-response relation such that higher cumulative exposures are associated with increased risk, then one may infer that persons who frequently enter the water and immerse their heads (for example, surfers) may have a higher risk of adverse health outcomes than the relatively infrequent swimmers included in this study.

In summary, we observed positive associations between adverse health effects and (1) distance from the drain, (2) bacterial indicators, and (3) presence of enteric viruses. Taken together, these results imply that there may be an increased risk of a broad range of adverse health effects associated with swimming in ocean water subject to urban runoff. Moreover, attributable numbers—that is, estimates of the number of new cases of an adverse health outcome that is attributable to the exposure of interest—reached well into the 100s per

10,000 exposed subjects for many of the positive associations observed here.[12] This finding implies that these risks might not be trivial when we consider the millions of persons who visit these beaches each year. Furthermore, the factors apparently contributing to the increased risk of adverse health outcomes observed here are not unique to Santa Monica Bay (similar levels of bacterial indicators are observed at many other beaches). Consequently, the prospect that untreated storm drain runoff poses a health risk to swimmers is probably relevant to many beaches subject to such runoff, including areas on the East, West, and Gulf coasts of North America, as well as numerous beaches on other continents.

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INVESTIGATION OF THE FEASIBILITY AND BENEFITS OF LOW-IMPACT SITE DESIGN PRACTICES ("LID") FOR VENTURA COUNTY

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ABSTRACT

The Clean Water Act NPDES permit that regulates municipal separate storm sewer systems (MS4s) in Ventura County, California will be reissued in 2007. The draft permit includes provisions for requiring the use of low impact development practices (LID) for certain kinds of development and redevelopment projects. Using six representative development project case studies, the author investigated the practicability and relative benefits of the permit's LID requirements. The results showed that (1) LID site design and source control techniques are more effective than conventional best management practices (BMPs) in reducing runoff rates; (2) Effective Impervious Area (EIA) can practicably be capped at three percent, a standard more protective than that proposed in the draft permit; and (3) in five out of six case studies, LID methods would reduce site runoff volume and pollutant loading to zero in typical rainfall scenarios.

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INTRODUCTION

The Assessment in Relation to Municipal Permit Conditions

This purpose of this study is to investigate the relative water quality and water reuse benefits of three levels of storm water treatment best management practices (BMPs): (1) basic "treat-and-release" BMPs (e.g., drain inlet filters, CDS units), (2) commonly used BMPs that expose runoff to soils and vegetation (extended-detention basins and biofiltration swales and filter strips), and (3) low-impact development (LID) practices. The factors considered in the investigation are runoff volume, pollutant loading, and the availability of water for infiltration or other reuse. In order to assess the differential impact of storm water reduction approaches on these factors, this study examines six case studies typical of development covered by the Ventura County Municipal Separate Storm Sewer System Permit.

Low-impact development methods reduce storm runoff and its contaminants by decreasing their generation at sources, infiltrating into the soil or evaporating storm flows before they can enter surface receiving waters, and treating flow remaining on the surface through contact with vegetation and soil, or a combination of these strategies. Soil-based LID practices often use soil enhancements such as compost, and thus improve upon the performance of more traditional basins and biofilters. For the study's purposes, verification of the practicability and utility of LID practices was based on a modified version of the Planning and Land Development Program (Part 4, section E) in the Draft Ventura County Municipal Separate Storm Sewer System Permit ("Draft Permit"). The Draft Permit requires that Effective Impervious Area (EIA) of certain types of new development and redevelopment projects be limited to five percent of total development project area. EIA is defined as hardened surface hydrologically connected via sheet flow or a discrete hardened conveyance to a drainage system or receiving water body. (Draft Permit p. 50) The study modified this requirement to three percent, as a way to test both the feasibility of meeting the higher, five percent standard in the draft permit and because as the lower, three percent EIA is essential to protect the Ventura County aquatic environment (see Attachment A).

The Draft Permit further requires minimizing the overall percentage of impervious surfaces in new development and redevelopment projects to support storm water infiltration. The Draft Permit also directs an integrated approach to minimizing and mitigating storm water pollution, using a suite of strategies including source control, LID, and treatment control BMPs. (Draft Permit p. 50) It is noted in this section of the document that impervious surfaces can be rendered "ineffective" if runoff is dispersed through properly designed vegetated swales. In testing the practicability of the draft permit's requirements and a three percent EIA standard, this study broadened this approach to encompass not only vegetated swales (channels for conveyance at some depth and velocity) but also vegetated filter strips (surfaces for conveyance in thin sheet flow) and bioretention areas (shallow basins with a range of vegetation types in which runoff infiltrates through soil either to groundwater or a subdrain for eventual surface discharge). The Draft Permit's stipulation of "properly designed" facilities was interpreted to entail, among other requirements, either determination that existing site soils can support runoff reduction through infiltration or that soils will be amended using accepted LID techniques to attain this objective. Finally, the study further broadened implementation options to include water harvesting (collection and storage for use in, for example, irrigation or gray water systems), roof downspout infiltration trenches, and porous pavements.

The Draft permit was interpreted to require management of EIA, other impervious area (what might be termed Not-Connected Impervious Area, NCIA), and pervious areas as follows:

- Runoff from EIA is subject to treatment control and the Draft Permit's Hydromodification Mitigation Control requirements before discharge.

- NCIA must be drained onto a properly designed vegetated surface or its runoff managed by one of the other options discussed in the preceding paragraph. To the extent NCIA runoff is not eliminated prior to discharge from the site in one of these ways, it is subject to treatment control and the Draft Permit's Hydromodification Mitigation Control requirements before discharge.

- Runoff from pervious areas is subject to treatment control and the Draft Permit's Hydromodification Mitigation Control requirements before discharge. This provision applies to pervious areas that both do and do not receive drainage from NCIA.

Where treatment control BMPs are required to manage runoff from the site, the Draft Permit's Volumetric or Hydrodynamic (Flow Based) Treatment Control design bases were assumed to apply. The former basis applies to storage-type BMPs, like ponds, and requires capturing and treating either the runoff volume from the 85th percentile 24-hour rainfall event for the location, the volume of annual runoff to achieve 80 percent or more volume treatment, or the volume of runoff produced from a 0.75 inch storm event. The calculations in this analysis used the 0.75-inch quantity. The Hydrodynamic basis applies to flow-through BMPs, like swales, and requires treating the runoff flow rate produced from a rain event equal to at least 0.2 inches per hour intensity (or one of two other approximately equivalent options).

Scope of the Assessment

With respect to each of the six development case studies, three assessments were undertaken: a baseline scenario incorporating no storm water management controls; a second scenario employing conventional BMPs; and a third development scenario employing LID storm water management strategies.

To establish a baseline for each case study, annual storm water runoff volumes were estimated, as well as concentrations and mass loadings of four pollutants: (1) total suspended solids (TSS), (2) total recoverable copper (TCu), (3) total recoverable zinc (TZn), and (4) total phosphorus (TP). These baseline estimates were based on the anticipated land use and cover with no storm water management efforts.

Two sets of calculations were then conducted using the parameters defined for the six case studies.

The first group of calculations estimated the extent to which basic BMPs reduce runoff volumes and pollutant concentrations and loadings, and what impact, if any, such BMPs have on recharge rates or water retention on-site.

The second group of calculations estimated the extent to which commonly used soil-based BMPs and LID site design strategies ameliorate runoff volumes and pollutant concentrations and loadings, and the effect such techniques have on recharge rates. When evaluating LID strategies, it was presumed that EIA would be limited to three percent and runoff from EIA, NCIA, and pervious areas would be managed as indicated above. The assessment of basins, biofiltration, and low-impact design practices analyzed the expected infiltration capacity of the case study sites. It also considered related LID techniques and practices, such as source reduction strategies, that could work in concert with infiltration to serve the goals of: (1) preventing increase in annual runoff volume from the pre- to the post-developed state, (2) preventing increase in annual pollutant mass loadings between the two development states, and (3) avoiding exceedances of California Toxics Rule (CTR) acute saltwater criteria for copper and zinc.

The results of this analysis show that:

- Developments implementing no post-construction BMPs result in storm water runoff volume and pollutant loading that are substantially increased, and recharge rates that are substantially decreased, compared to pre-development conditions.

- Developments implementing basic post-construction treatment BMPs achieve reduced pollutant loading compared to developments with no BMPs, but storm water runoff volume and recharge rates are similar to developments with no BMPs.

- Developments implementing traditional basins and biofilters, and even more so low-impact post-construction BMPs, achieve significant reduction of pollutant loading and runoff volume as well as greatly enhanced recharge rates compared to both developments with no BMPs and developments with basic treatment BMPs.

- Typical development categories, ranging from single family residential to large commercial, can feasibly implement low-impact post-construction BMPs designed in compliance with the draft permit's requirements, as modified to include a lower, three percent EIA requirement.

This report covers the methods employed in the investigation, data sources, and references for both. It then presents the results, discusses their consequences, draws conclusions, and makes recommendations relative to the feasibility of utilizing low-impact development practices in Ventura County developments.

CASE STUDIES

Six case studies were selected to represent a range of urban development types considered to be representative of coastal Southern California, including Ventura County. These case studies involved: a multi-family residential complex (MFR), a relatively small-scale (23 homes) single-family residential development (Sm-SFR), a restaurant (REST), an office building (OFF), a relatively large (1000 homes) single-family residential development (Lg-SFR) and a sizeable commercial retail installation (COMM).[1]

Parking spaces were estimated to be 176 sq ft in area, which corresponds to 8 ft width by 22 ft length dimensions. Code requirements vary by jurisdiction, with the tendency now to drop below the traditional 200 sq ft average. About 180 sq ft is common, but various standards for full- and compact-car spaces, and for the mix of the two, can raise or lower the average.[2] The 176 sq ft size is considered to be a reasonable value for conventional practice.

Roadways and walkways assume a wide variety of patterns. Exclusive of the two SFR cases, simple, square parking lots with roadways around the four sides and square buildings with walkways also around the four sides were assumed. Roadways and walkways were taken to be 20 ft and 6 ft wide, respectively.

Single-family residences were assumed each to have a driveway 20 ft wide and 30 ft long. It was further assumed that each would have a sidewalk along the front of the lot, which was calculated to be 5749 sq ft in area. Assuming a square lot, the front dimension would be 76 ft. A 40-ft walkway was included within the property. Sidewalks and walkways were taken to be 4 ft wide.

Exclusive of the COMM case, the total area for all of these impervious features was subtracted from the total site area to estimate the pervious area, which was assumed to have conventional landscaping cover (grass, small herbaceous decorative plants, bushes, and a few trees). For the COMM scenario, the hypothetical total impervious cover was enlarged by 10 percent to represent the landscaping, on the belief that a typical retail commercial establishment would typically be mostly impervious.

Table 1 (page 5) summarizes the characteristics of the six case studies. The table also provides the recorded or estimated areas in each land use and cover type.

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[1] Building permit records from the City of San Marcos in San Diego County provided data on total site areas for the first four case studies, including numbers of buildings, building footprint areas (including porch and garage for Sm-SFR), and numbers of parking spaces associated with the development projects. While the building permit records made no reference to features such as roadways, walkways, and landscaping normally associated with development projects, these features were taken into account in the case studies using assumptions described herein. Larger developments were not represented in the sampling of building permits from the San Marcos database. To take larger development projects into account in the subsequent analysis, the two larger scale case studies were hypothesized. The Lg-SFR scenario scaled up all land use estimates from the Sm-SFR case in the ratio of 1000:23. The hypothetical COMM scenario consisted of a building with a 2-acre footprint and 500 parking spaces. As with the smaller-scale cases, these hypothetical developments were assumed to have roadways, walkways, and landscaping, as described herein.

[2] J. Gibbons, *Parking Lots*, NONPOINT EDUCATION FOR MUNICIPAL OFFICERS, Technical Paper No. 5 (1999) (http://nemo.uconn.edu/tools/publications/tech_papers/tech_paper_5.pdf).

Table 1. Case Study Characteristics and Land Use and Land Cover Areas

	MFR[a]	Sm-SFR[a]	REST[a]	OFF[a]	Lg-SFR[a]	COMM[a]
No. buildings	11	23	1	1	1000	1
Total area (ft^2)	476,982	132,227	33,669	92,612	5,749,000	226,529
Roof area (ft^2)	184,338	34,949	3,220	7,500	1,519,522	87,120
No. parking spaces	438	-	33	37	-	500
Parking area (ft^2)	77,088	-	5808	6512	-	88,000
Access road area (ft^2)	22,212	-	6097	6456	-	23,732
Walkway area (ft^2)	33,960	10,656	1362	2078	463,289	7,084
Driveway area (ft^2)	-	13,800	-	-	600,000	-
Landscape area (ft^2)	159,384	72,822	17,182	70,066	3,166,190	20,594

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential;
REST—restaurant; OFF—office building; Lg-SFR—large-scale single-family residential; COMM—retail commercial

METHODS OF ANALYSIS

Annual Storm Water Runoff Volumes

Annual surface runoff volumes produced were estimated for both pre- and post-development conditions for each case study site. Runoff volume was computed as the product of annual precipitation, contributing drainage area, and a runoff coefficient (ratio of runoff produced to rainfall received). For impervious areas the following equation was used:

$$\mathbf{C = (0.009)\,\mathit{I} + 0.05}$$

where I is the impervious percentage. This equation was derived by Schueler (1987) from Nationwide Urban Runoff Program data (U.S. Environmental Protection Agency 1983). With I = 100 percent for fully impervious surfaces, C is 0.95.

The basis for pervious area runoff coefficients was the Natural Resource Conservation Service's (NRCS) Urban Hydrology for Small Watersheds (NRCS 1986, as revised from the original 1975 edition). This model estimates storm event runoff as a function of precipitation and a variable representing land cover and soil, termed the curve number (CN). Larger events are forecast to produce a greater amount of runoff in relation to amount of rainfall because they more fully saturate the soil. Therefore, use of the model to estimate annual runoff requires selecting some event or group of events to represent the year. A 0.75-inch rainfall event was used in the analysis here for the relative comparison between pre- and post-development and applied to deriving a runoff coefficient for annual estimates, recognizing that smaller storms would produce less and larger storms more runoff.

To select CN for the pre-development case, an analysis performed in the area of the Cedar Fire in San Diego County was used in which CN was determined before and after the 2003 fire.[3] In the San Diego analysis, CN = 83 was estimated for the pre-existing land cover, which was generally chaparral, a vegetative cover also typical of Ventura County. As indicated below, soils are also similar in Ventura and San Diego Counties, making the parameter selection reasonable for use in both locations. For post-development landscaping, CN = 86 was selected based on tabulated data in NRCS (1986) and professional judgment.

Pre- and post-development runoff quantities were computed with these CN values and the 0.75-inch rainfall, and then divided by the rainfall to obtain runoff coefficients. The results were 0.07

[3] American Forests, *San Diego Urban Ecosystem Analysis After the Cedar Fire* (Feb. 3, 2006) (http://www.ufei.org/files/pubs/SanDiegoUrbanEcosystemAnalysis-PostCedarFire.pdf).

and 0.12, respectively. Finally, total annual runoff volumes were estimated based on an average annual precipitation in the City of Ventura of 14.71 inches.[4]

Storm Water Runoff Pollutant Discharges

Annual pollutant mass discharges were estimated as the product of annual runoff volumes produced by the various land use and cover types and pollutant concentrations typical of those areas. Again, the 0.75-inch precipitation event was used as a basis for volumes. Storm water pollutant data have typically been measured and reported for general land use types (e.g., single-family residential, commercial). However, an investigation of low-impact development practices of the type this study sought to conduct demands data on specific land coverages. The literature offers few data on this basis. Those available and used herein were assembled by a consultant to the City of Seattle for a project in which the author participated. They appear in Attachment B (Herrera Environmental Consultants, Inc. undated).

Pollutant concentrations expected to occur typically in the mixed runoff from the several land use and cover types making up a development were estimated by mass balance; i.e., the concentrations from the different areas of the sites were combined in proportion to their contribution to the total runoff.

The Effect of Conventional Treatment BMPs on Runoff Volume, Pollutant Discharges, and Recharge Rates

The first question in analyzing how BMPs reduce runoff volumes and pollutant discharges was, What BMPs are being employed in Ventura County developments under the permit now in force? This permit is open-ended and provides regulated entities with a large number of choices and few fixed requirements. These options presumably include manufactured BMPs, such as drain inlet inserts (DIIs) and continuous deflective separation (CDS) units. Developments may also select such non-proprietary devices as extended-detention basins (EDBs) and biofiltration swales and filter strips. EDBs hold water for two to three days for solids settlement before releasing whatever does not infiltrate or evaporate. Biofiltration treats runoff through various processes mediated by vegetation and soil. In a swale, runoff flows at some depth in a channel, whereas a filter strip is a broad surface over which water sheet flows. Each of these BMP types was applied to each case study, although it is not clear that these BMPs, in actuality, have been implemented consistently within Ventura County to date.

The principal basis for the analysis of BMP performance was the California Department of Transportation's (CalTrans, 2004) BMP Retrofit Pilot Program, performed in San Diego and Los Angeles Counties. One important result of the program was that BMPs with a natural surface infiltrate and evaporate (probably, mostly infiltrate) a substantial amount of runoff, even if conditions do not appear to be favorable for an infiltration basin. On average, the EDBs, swales, and filter strips lost 40, 50 and 30 percent, respectively, of the entering flow before the discharge point. DIIs and CDS units do not contact runoff with a natural surface, and therefore do not reduce runoff volume.

The CalTrans program further determined that BMP effluent concentrations were usually a function of the influent concentrations, and equations were developed for the functional

[4] Ventura County Watershed Protection District (http://www.vcwatershed.org/fws/specialmedia.htm). The City of Ventura is considered to be representative of most of the developed and developing areas in Ventura County. However, there is some variation around the county, with the maximum precipitation registered at Ojai (annual average 21.32 inches). Ojai is about 15 miles inland and lies at elevation 745 ft at the foot of the Topatopa Mountains, the orographic effect of which influences its meteorology. Ojai's higher rainfall was taken into account in the calculations, and the report notes the few instances where it affected the conclusions.

relationships in these cases. BMPs generally reduced influent concentrations proportionately more when they were high. In relatively few situations influent concentrations were constant at an "irreducible minimum" level regardless of inflow concentrations.

In analyzing the effects of BMPs on the case study runoff, the first step was to reduce the runoff volumes estimated with no BMPs by the fractions observed to be lost in the pilot study. The next task was estimating the effluent concentrations from the relationships in the CalTrans report. The final step was calculating discharge pollutant loadings as the product of the reduced volumes and predicted effluent concentrations. As before, typical pollutant concentrations in the mixed runoff were established by mass balance.

Estimating Infiltration Capacity of the Case Study Sites

Infiltrating sufficient runoff to maintain pre-development hydrologic characteristics and prevent pollutant transport is the most effective way to protect surface receiving waters. Successfully applying infiltration requires soils and hydrogeological conditions that will pass water sufficiently rapidly to avoid overly-lengthy ponding, while not allowing percolating water to reach ground-water before the soil column captures pollutants.

The study assumed that infiltration would occur in surface facilities and not in below-ground trenches. The use of trenches is certainly possible, and was judged to be an approved BMP by CalTrans after the pilot study. However, the intent of this investigation was to determine the ability of pervious areas to manage the site runoff. This was accomplished by determining the infiltration capability of the pervious areas in their original condition for each development case study, and further assessing the pervious areas' infiltration capabilities if soils were modified according to low impact development practices.

The chief basis for this aspect of the work was an assessment of infiltration capacity and benefits for Los Angeles' San Fernando Valley (Chralowicz et al. 2001). The Chralowicz study posited providing 0.1-0.5 acre for infiltration basins to serve each 5 acres of contributing drainage area. At 2-3 ft deep, it was estimated that such basins could infiltrate 0.90-1.87 acre-ft/year of runoff in San Fernando Valley conditions. Soils there are generally various loam textures with infiltration rates of approximately 0.5-2.0 inches/hour. The most prominent soils in Ventura County, at least relatively near the coast, are loams, sandy loams, loamy sands, and silty clay loams, thus making the conclusions of the San Fernando Valley study applicable for these purposes.[5] This information was used to estimate how much of each case study site's annual runoff would be infiltratable, and if the pervious portion would provide sufficient area for infiltration. For instance, if sufficient area were available, the infiltration configuration would not have to be in basin form but could be shallower and larger in surface area. This study's analyses assumed the use of bioretention areas rather than traditional infiltration basins.

Volume and Pollutant Source Reduction Strategies

As mentioned above, the essence of low-impact development is reducing runoff problems before they can develop, at their sources, or exploiting the infiltration and treatment abilities of soils and vegetation. If a site's existing infiltration and treatment capabilities are inadequate to preserve pre-development hydrology and prevent runoff from causing or contributing to violations of water quality standards, then LID-based source reduction strategies can be implemented, infiltration and treatment capabilities can be upgraded, or both.

[5] Cabrillo Port Liquefied Natural Gas Deepwater Port Draft EIS/EIR (Oct. 2004) (http://www.cabrilloport.ene.com/files/eiseir/4.05%20%20-Agriculture%20and%20Soils.pdf).

Source reduction can be accomplished through various LID techniques. Soil can be upgraded to store runoff until it can infiltrate, evaporate, or transpire from plants through compost addition. Soil amendment, as this practice is known, is a standard LID technique.

Upgraded soils are used in bioretention cells that hold runoff and effect its transfer to the subsurface zone. This standard LID tool can be used where sufficient space is available. This study analyzed whether the six development case study sites would have sufficient space to effectively reduce runoff using bioretention cells, assuming the soils and vegetation could be amended and enhanced where necessary.

Conventional pavements can be converted to porous asphalt or concrete or replaced with concrete or plastic unit pavers or grid systems. For such approaches to be most effective, the soils must be capable of infiltrating the runoff passing through, and may require renovation.

Source reduction can be enhanced by the LID practice of water harvesting, in which water from impervious surfaces is captured and stored for reuse in irrigation or gray water systems. For example, runoff from roofs and parking lots can be harvested, with the former being somewhat easier because of the possibility of avoiding pumping to use the water and fewer pollutants. Harvesting is a standard technique for Leadership in Energy and Environmental Design (LEED) buildings.[6] Many successful systems of this type are in operation, such as the Natural Resources Defense Council offices (Santa Monica, CA), the King County Administration Building (Seattle, WA), and two buildings on the Portland State University campus (Portland, OR). This investigation examined how water harvesting could contribute to storm water management for case study sites where infiltration capacity, available space, or both appeared to be limited.

RESULTS OF THE ANALYSIS

1. "Base Case" Analysis: Development without Storm Water Controls

Comparison of Pre- and Post-Development Runoff Volumes

Table 2 (page 9) presents a comparison between the estimated runoff volumes generated by the respective case study sites in the pre- and post-development conditions, assuming implementation of no storm water controls on the developed sites. On sites dominated by impervious land cover, most of the infiltration that would recharge groundwater in the undeveloped state is expected to be lost to surface runoff after development. This greatly increased surface flow would raise peak flow rates and volumes in receiving water courses, raise flooding risk, and transport pollutants. Only the office building, the plan for which retained substantial pervious area, would lose less than half of the site's pre-development recharge.

[6] New Buildings Institute, Inc., *Advanced Buildings* (2005) (http://www.poweryourdesign.com/LEEDGuide.pdf).

Table 2. Pre- and Post-Development without BMPs: Distribution of Surface Runoff Versus Recharge to Groundwater

Annual Volume (acre-ft)	MFR[a]	Sm-SFR[a]	REST[a]	OFF[a]	Lg-SFR[a]	COMM[a]
Precipitation[b]	13.4	3.72	0.95	2.60	162	6.37
Pre-development runoff[c]	0.94	0.26	0.07	0.18	11	0.45
Pre-development recharge[d]	12.5	3.46	0.88	2.42	150	5.92
Post-development impervious runoff[c]	8.48	1.59	0.44	0.60	69	5.50
Post-development pervious runoff[c]	0.54	0.25	0.06	0.24	11	0.07
Post-development total runoff[c]	9.02	1.83	0.50	0.84	80	5.57
Post-development recharge[d]	4.39	1.88	0.45	1.76	82	0.80
Post-development recharge loss (% of pre-development recharge)	8.08 (65%)	1.57 (46%)	0.43 (49%)	0.66 (27%)	68 (45%)	5.12 (86%)

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential; REST—restaurant; OFF—office building; Lg-SFR—large-scale single-family residential;
COMM—retail commercial
[b] Volume of precipitation on total project area
[c] Quantity of water discharged from the site on the surface
[d] Quantity of water infiltrating the soil; the difference between precipitation and runoff

Pollutant Concentrations and Loadings

Table 3 presents the pollutant concentrations from the literature and loadings calculated as described for the various land use and cover types represented by the case studies. Landscaped areas are expected to release the highest TSS concentration, although relatively low TSS mass loading because of the low runoff coefficient. The highest copper concentrations and loadings are expected from parking lots. Roofs, especially commercial roofs, top the list for both zinc concentrations and loadings. Landscaping would issue by far the highest phosphorus, although access roads and driveways would contribute the highest mass loadings.

Table 3. Pollutant Concentrations and Loadings for Case Study Land Use and Cover Types

Land Use	Concentrations				Loadings			
	TSS (mg/L)	TCu (mg/L)	TZn (mg/L)	TP (mg/L)	Lbs. TSS/ acre-year	Lbs. TCu/ acre-year	Lbs. TZn/ acre-year	Lbs. TP/ acre-year
Residential roof	25	0.013	0.159	0.11	79	0.041	0.503	0.348
Commercial roof	18	0.014	0.281	0.14	57	0.044	0.889	0.443
Access road/driveway	120	0.022	0.118	0.66	380	0.070	0.373	2.088
Parking	75	0.036	0.097	0.14	237	0.114	0.307	0.443
Walkway	25	0.013	0.059	0.11	79	0.041	0.187	0.348
Landscaping	213	0.013	0.059	2.04	85	0.005	0.024	0.815

The CTR acute criteria for copper and zinc are 0.0048 mg/L and 0.090 mg/L, respectively. Table 3 shows that all developed land uses are expected to discharge copper above the criterion, based on the mass balance calculations using concentrations from Table 3. Any surface release from the case study sites would violate the criterion at the point of discharge, although dilution by the receiving water would lower the concentration below the criterion at some point. Even if copper mass loadings are reduced by BMPs, any surface discharge would exceed the criterion initially, but it would be easier to dilute below that level. In contrast, runoff from some land covers would not violate the acute zinc criterion. Because of this difference, the evaluation considered whether or not the zinc criterion would be exceeded in each analysis, whereas there was no point in this analysis for copper. There are no equivalent water quality criteria for TSS and TP; hence, their concentrations were not further analyzed in the different scenarios.

Table 4 shows the overall loadings, as well as zinc concentrations, expected to be delivered from the case study developments should they not be fitted with any BMPs. As Table 4 shows, all cases are forecast to exceed the 0.090 mg/L acute zinc criterion, and the retail commercial development does so by a wide margin. Because of its size, the large residential development dominates the mass loading emissions.

Table 4. Case Study Pollutant Concentration and Loading Estimates without BMPs

	MFR[a]	Sm-SFR[a]	REST[a]	OFF[a]	Lg-SFR[a]	COMM[a]
TZn (mg/L)	0.127	0.123	0.128	0.133	0.123	0.175
Lbs. TSS/year	1321	345	125	242	15016	853
Lbs. TCu/year	0.46	0.074	0.032	0.045	3.21	0.37
Lbs. TZn/year	3.09	0.607	0.174	0.301	26.4	2.64
Lbs. TP/year	6.58	2.39	0.72	1.78	104	3.36

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential; REST—restaurant; OFF—office building; Lg-SFR—large-scale single-family residential; COMM—retail commercial

2. "Conventional BMP" Analysis: Effect of Basic Treatment BMPs

Effect of Basic Treatment BMPs on Post-Development Runoff Volumes

The current permit allows regulated parties to select from a range of BMPs in order to treat or infiltrate a given quantity of annual rainfall. The range includes drain inlet inserts, CDS units, and other manufactured BMPs, detention vaults, and sand filters, all of which isolate runoff from the soil; as well as basins and biofiltration BMPs built in soil and generally having vegetation. Treatment BMPs that do not permit any runoff contact with soils discharge as much storm water runoff as equivalent sites with no BMPs, and hence yield zero savings in recharge. As mentioned above, the CalTrans (2004) study found that BMPs with a natural surface can reduce runoff by substantial margins (30-50 percent for extended-detention basins and biofiltration).

With such a wide range of BMPs in use, runoff reduction ranging from 0 to 50 percent, and a lack of clearly ascertainable requirements, it is not possible to make a single estimate of how much recharge savings are afforded by maximal implementation of the current permit. We made the following assumptions regarding implementation of BMPs. Assuming natural-surface BMPs perform at the average of the three types tested by CalTrans (2004), i.e., 40 percent runoff reduction, the estimate can be bounded as shown in Table 5 (page 11). The table demonstrates that allowing free choice of BMPs without regard to their ability to direct water into the ground forfeits substantial groundwater recharge benefits when hardened-surface BMPs are selected. Use of soil-based conventional BMPs could cut recharge losses from half or e more of the full potential to about one-quarter to one-third or less, except with the highly impervious commercial development. This analysis shows the wisdom of draining impervious to pervious surfaces, even if those surfaces are not prepared in any special way. But as subsequent analyses showed, soil amendment can gain considerably greater benefits.

Table 5. Pre- and Post-Development with Conventional BMPs: Distribution of Surface Runoff Versus Recharge to Groundwater

Annual Volume (acre-ft)	MFR[a]	Sm-SFR[a]	REST[a]	OFF[a]	Lg-SFR[a]	COMM[a]
Precipitation[b]	13.4	3.72	0.95	2.60	162	6.37
Pre-development runoff[c]	0.94	0.26	0.07	0.18	11	0.45
Pre-development recharge	12.5	3.46	0.88	2.42	150	5.92
Post-development impervious runoff[c, d]	5.09-8.48	0.95-1.59	0.26-0.44	0.36-0.60	41-69	3.30-5.50
Post-development pervious runoff[c, d]	0.32-0.54	0.15-0.25	0.04-0.06	0.14-0.24	6.6-11	0.04-0.07
Post-development total runoff[c, d]	5.41-9.02	1.10-1.83	0.30-0.50	0.50-0.84	48-80	3.34-5.57
Post-development recharge[d, e]	4.39-7.99	1.88-2.62	0.45-0.65	1.76-2.10	82-114	0.80-3.03
Post-development recharge loss (% of pre-development recharge) [d, e]	4.51-8.08 (36-65%)	0.84-1.57 (24-46%)	0.23-0.43 (26-49%)	0.32-0.66 (13-27%)	36-68 (24-45%)	2.89-5.12 (49-86%)

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential; REST—restaurant; OFF—office building; Lg-SFR—large-scale single-family residential; COMM—retail commercial. Ranges represent 40 percent runoff volume reduction, with full site coverage by BMPs having a natural surface, to no reduction, with BMPs isolating runoff from soil.
[b] Volume of precipitation on total project area
[c] Quantity of water discharged from the site on the surface
[d] Ranging from the quantity with hardened bed BMPs to the quantity with soil-based BMPs
[e] Quantity of water infiltrating the soil; the difference between precipitation and runoff

Effect of Basic Treatment BMPs on Pollutant Discharges

Table 6 (page 12) presents estimates of zinc effluent concentrations and mass loadings of the various pollutants discharged from four types of conventional treatment BMPs. The manufactured CDS BMPs in this table, which do not expose runoff to soil or vegetation, are not expected to drop any of the concentrations sufficiently to meet the acute zinc criterion at the discharge point. The loading reduction results show the CDS units always performing below 50 percent reduction for all pollutants analyzed, and most often in the vicinity of 20 percent, with zero copper reduction.

When treated with swales or filter strips, effluents from each development case study site are expected to fall below the CTR acute zinc criterion. All but the large commercial site would meet the criterion with EDB treatment. These natural-surface BMPs, if fully implemented and well maintained, are predicted to prevent the majority of the pollutant masses generated on most of the development sites from reaching a receiving water. Only total phosphorus reduction falls below 50 percent for two case studies. Otherwise, mass loading reductions range from about 60 to above 80 percent for the EDB, swale, and filter strip. This data indicates that draining impervious to pervious surfaces, even if those surfaces are not prepared in any special way, pays water quality as well as hydrologic dividends.

Table 6. Pollutant Concentration and Loading Reduction Estimates with Conventional BMPs

	MFR[a]	Sm-SFR[a]	REST[a]	OFF[a]	Lg-SFR[a]	COMM[a]
Effluent Concentrations:						
CDS TZn (mg/L)[a]	0.095	0.095	0.098	0.102	0.095	0.131
EDB TZn (mg/L)[a]	0.085	0.086	0.084	0.084	0.086	0.098
Swale TZn (mg/L)	0.055	0.054	0.055	0.056	0.054	0.068
Filter strip TZn (mg/L)	0.039	0.039	0.039	0.041	0.039	0.048
Loading Reductions:						
CDS TSS loading reduction	15.7%	19.9%	22.0%	24.0%	19.9%	16.9%
CDS TCu loading reduction	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%
CDS TZn loading reduction	22.7%	22.4%	22.9%	23.1%	22.4%	25.1%
CDS TP loading reduction	30.6%	41.5%	40.7%	45.9%	41.5%	20.3%
EDB TSS loading reduction	68.1%	73.7%	79.0%	81.1%	73.7%	71.7%
EDB TCu loading reduction	61.9%	55.7%	66.2%	63.0%	55.7%	66.8%
EDB TZn loading reduction	59.7%	59.6%	60.4%	61.9%	59.6%	66.6%
EDB TP loading reduction	61.9%	69.7%	69.1%	72.9%	69.7%	54.5%
Swale TSS loading reduction	68.8%	71.1%	73.1%	73.9%	71.1%	69.4%
Swale TCu loading reduction	72.5%	68.5%	78.2%	73.3%	68.5%	75.8%
Swale TZn loading reduction	78.4%	78.1%	84.3%	78.8%	78.1%	80.7%
Swale TP loading reduction	66.3%	70.7%	67.2%	76.2%	70.7%	55.0%
Filter strip TSS loading reduction	69.9%	75.4%	80.6%	82.6%	75.4%	72.3%
Filter strip TCu loading reduction	74.4%	69.1%	78.2%	75.4%	69.1%	78.7%
Filter strip TZn loading reduction	78.3%	77.9%	78.4%	78.7%	77.9%	80.9%
Filter strip TP loading reduction	48.4%	53.1%	63.7%	59.8%	53.1%	34.6%

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential; REST—restaurant; OFF—office building; Lg-SFR—large-scale single-family residential; COMM—retail commercial; CDS— continuous deflective separation unit; EDB—extended-detention basin

3. LID Analysis: Development According to Modified Draft Permit Provisions

(a) Hydrologic Analysis

The LID analysis was first performed according to the Draft Permit provisions under the Planning and Land Development Program (Part 4, section E). In this analysis, however, EIA was limited to three instead of five percent, under the reasoning presented in Attachment A. All runoff from NCIA was assumed to drain to vegetated surfaces, as provided in the Draft Permit.

One goal of this exercise was to identify methods that reduce runoff production in the first place. It was hypothesized that implementation of source reduction techniques could allow all of the case study sites to infiltrate substantial proportions of the developed site runoff, advancing the hydromodification mitigation objective of the Draft Permit. When runoff is dispersed into the soil instead of being rapidly collected and conveyed away, it recharges groundwater, supplementing a resource that maintains dry season stream flow and wetlands. An increased water balance can be tapped by humans for potable, irrigation, and process water supply. Additionally, runoff volume reduction would commensurately decrease pollutant mass loadings.

Accordingly, the analysis considered the practicability of more than one scenario by which the draft permit's terms could be met, as modified to reflect three percent EIA. In one option, all roof runoff is harvested and stored for some beneficial use. A second option disperses runoff into the soil via roof downspout infiltration trenches. The former option is probably best suited to cases like the large commercial and office buildings, while distribution in the soil would fit best with residences and relatively small commercial developments. The analysis was repeated with the assumptions of harvesting OFF and COMM roof runoff for some beneficial use and dispersing roof runoff from the remaining four cases in roof downspout infiltration systems.

Expected Infiltration Capacities of the Case Study Sites

The first inquiry on this subject sought to determine how much of the total annual runoff each property is expected to infiltrate. This assessment tested the feasibility of draining all but three percent of impervious area to pervious land on the sites. Based on the findings of Chralowicz et al. (2001), it was assumed that an infiltration zone of 0.1-0.5 acres in area and 2-3 ft deep would serve a drainage catchment area in the size range 0-5 acres and infiltrate 0.9-1.9 acre-ft/year. The conclusions of Chralowicz et al. (2001) were extrapolated to conservatively assume that 0.5 acre would be required to serve each additional five acres of catchment, and would infiltrate an incremental 1.4 acre-ft/year (the midpoint of the 0.9-1.9 acre-ft/year range). According to these assumptions, the following schedule of estimates applies:

Pervious Area Available for Infiltration	Catchment Served acres	Infiltration Capacity
0.5 acres	0-5 acres	1.4 acre-ft/year
1.0 acres	5-10 acres	2.8 acre-ft/year
1.5 acres	10-15 acres	4.2 acre-ft/year
(Etc.)	...	...

As a formula, infiltration capacity ≈ 2.8 x available pervious area. To apply the formula conservatively, the available area was reduced to the next lower 0.5-acre increment before multiplying by 2.8.

As shown in Table 7, five of the six sites have adequate or greater capacity to infiltrate the full annual runoff volume from NCIA and pervious areas where EIA is limited to three percent of the total site area (four at the higher Ojai rainfall). Indeed, five of the six development types have sufficient pervious area to infiltrate *all* runoff, including runoff from EIA areas. With the most representative rainfall, only the large commercial development, with little available pervious area, falls short of the needed capacity to infiltrate all rainfall, but it still has the capacity to meet the terms of the draft permit, as modified for this analysis. These results are based on infiltrating in the native soils with no soil amendment. For any development project at which infiltration-oriented BMPs are considered, it is important that infiltration potential be carefully assessed using site-specific soils and hydrogeologic data. In the event such an investigation reveals a marginal condition (e.g., hydraulic conductivity, spacing to groundwater) for infiltration basins, soils could be enhanced to produce bioretention zones to assist infiltration. Notably, the four case studies with far greater than necessary infiltration capacity would offer substantial flexibility in designing infiltration, allowing ponding at less than 2-3 ft depth.

Table 7. Infiltration and Runoff Volume With 3 Percent EIA and All NCIA Draining to Pervious Areas

	MFR[a]	Sm-SFR[a]	REST[a]	OFF[a]	Lg-SFR[a]	COMM[a]
EIA runoff (acre-ft/year)	0.38	0.11	0.03	0.07	4.6	0.18
NCIA + pervious area runoff (acre-ft/year)	**8.63**	**1.73**	**0.47**	**0.76**	**75.0**	**5.39**
Total runoff (acre-ft/year)	9.01	1.84	0.50	0.83	79.6	5.57
Pervious area available for infiltration (acres)	3.66	1.67	0.39	1.61	72.7	0.47
Estimated infiltration capacity (acre-ft/year)[b]	**9.8**	**4.2**	**1.4**	**4.2**	**203**	**1.4**
Infiltration capacity [c]	> 100%[d]	> 100%	> 100%	> 100%	> 100%	~26% [d]

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential; REST—restaurant; OFF—office building; Lg-SFR—large-scale single-family residential; COMM—retail commercial;
[b] Based on Chralowicz et al. (2001) according to the schedule described above
[c] Compare runoff production from NCIA + pervious area (**row 3**) with estimated infiltration capacity (**row 6**)
[d] At Ojai rainfall levels, capacity would be ~78 percent at the MFR site and ~18 percent at the COMM site.

As Table 7 shows, five of the six case study sites have the capacity to infiltrate *all* runoff produced onsite by draining impervious surfaces to pervious areas. Even runoff from the area assumed to be EIA could be infiltrated in most cases based on the amount of pervious area available in typical development projects. By showing that it is possible under normal site conditions and using native soils to retain *all* runoff in typical developments, these results demonstrate that a three percent EIA requirement, which would not demand that all runoff be retained, is feasible and practicable.

Additional Source Reduction Capabilities of the Case Study Sites: Water Harvesting Example

Infiltration is one of a wide variety of LID-based source reduction techniques. Where site conditions such as soil quality or available area limit a site's infiltration capacity, other source LID measures can enhance a site's runoff retention capability. For example, soil amendment, which improves infiltration, is a standard LID technique. Water harvesting is another. Such practices can also be used where infiltration capacity is adequate, but the developer desires greater flexibility for land use on-site. Table 8 shows the added implementation flexibility created by subtracting roof runoff by harvesting it or efficiently directing it into the soil through downspout dispersion systems, further demonstrating the feasibility of meeting the draft permit's proposed requirements, as modified to include a three percent EIA standard.

Table 8. Infiltration and Runoff Volume Reduction Analysis Including Roof Runoff Harvesting or Disposal in Infiltration Trenches (Assuming 3 Percent EIA and All NCIA Draining to Pervious Areas)

	MFR[a]	Sm-SFR[a]	REST[a]	OFF[a]	Lg-SFR[a]	COMM[a]
EIA runoff (acre-ft/year)	0.38	0.11	0.03	0.07	4.6	0.18
Roof runoff (acre-ft/year)	4.92	0.93	0.09	0.20	41	2.33
Other NCIA + pervious area runoff (acre-ft/year)	**3.71**	**0.79**	**0.39**	**0.56**	**35**	**3.06**
Total runoff (acre-ft/year)	9.01	1.84	0.50	0.83	79.6	5.57
Pervious area available for infiltration (acres)	3.66	1.67	0.39	1.61	72.7	0.47
Estimated infiltration capacity (acre-ft/year)[b]	**9.8**	**4.2**	**1.4**	**4.2**	**203**	**1.4**
Infiltration capacity [c]	> 100%	> 100%	> 100%	> 100%	> 100%	~45% [d]

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential; REST—restaurant; OFF—office building; Lg-SFR—large-scale single-family residential; COMM—retail commercial;
[b] Based on Chralowicz et al. (2001) according to the schedule described above
[c] Comparison of runoff production from NCIA + pervious area (**row 3**) with estimated infiltration capacity (**row 6**)
[d] If the higher rainfall at Ojai is assumed, capacity would be ~32 percent of the amount needed for the COMM case.

Effect of Full LID Approach on Recharge

Table 9 (page 15) shows the recharge benefits of preventing roofs from generating runoff and infiltrating as much as possible of the runoff from the remainder of the case study sites. The data show that LID methods offer significant benefits relative to the baseline (no storm water controls) in all cases. These benefits are particularly impressive in developments with relatively high site imperviousness, such as in the MFR and COMM cases. In the latter case the full LID approach (excluding the common and effective practice of soil amendment) would cut loss of the potential water resource represented by recharge and harvesting from 86 to 37 percent.

Table 9. Comparison of Water Captured Annually (in acre-ft) from Development Sites for Beneficial Use With a Full LID Approach Compared to Development With No BMPs

	MFR[a]	Sm-SFR[a]	REST[a]	OFF[a]	Lg-SFR[a]	COMM[a]
Pre-development recharge[b] (acre-ft)	12.5	3.46	0.88	2.42	150	5.92
No BMPs:						
post-development recharge [b] (acre-ft)	4.39	1.88	0.45	1.76	82	0.80
post-development runoff (acre-ft)	8.08	1.57	0.43	0.66	68	5.12
post-development % recharge lost	65%	46%	49%	27%	45%	86%
Full LID approach:						
post-development runoff capture (acre-ft)[c]	12.5	3.46	0.88	2.42	150	3.73
post-development runoff (acre-ft)	0	0	0	0	0	2.19
post-development % recharge lost	0%	0%	0%	0%	0%	37%

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential; REST—restaurant; OFF—office building; Lg-SFR—large-scale single-family residential; COMM—retail commercial
[b] Quantity of water infiltrating the soil; the difference between precipitation and runoff
[c] Water either entirely infiltrated in BMPs and recharged to groundwater or partially harvested from roofs and partially infiltrated in BMPs. For the first five case studies, EIA was not distinguished from the remainder of the development, because these sites have the potential to capture all runoff.

(b) Water Quality Analysis

As outlined above, it was assumed that EIA discharges, as well as runoff from all pervious surfaces, are subject to treatment control. For purposes of the analysis, treatment control was assumed to be provided by conventional sand filtration. This choice is appropriate for study purposes for two reasons. First, sand filters can be installed below grade, and land above can be put to other uses. Under the Draft Permit's approach, pervious area should be reserved for receiving NCIA drainage, and using sand filters would not draw land away from that service or other site uses. A second reason for the choice is that sand filter performance data equivalent to the data used in analyzing other conventional BMPs are available from the CalTrans (2004) work. Sand filters may or may not expose water to soil, depending on whether or not they have a hard bed. This analysis assumed a hard bed, meaning that no infiltration would occur and thus there would be no additional recharge in sand filters. Performance would be even better than shown in the analytical results if sand filters were built in earth.

Pollutant Discharge Reduction Through LID Techniques

The preceding analyses demonstrated that each of the six case studies could feasibly comply with the draft permit's requirements, as modified to include a more protective three percent EIA standard. Moreover, for five of the six case studies, *all* storm water discharges could be eliminated at least under most meteorological conditions by dispersing runoff from impervious surfaces to pervious areas. Therefore, pollutant additions to receiving waters would also be eliminated. This demonstrates not only that a lower EIA (three percent) is a feasible and practicable approach to maintaining the natural hydrology of land being developed, as discussed above, but that a lower EIA is a feasible and practicable way to eliminate the discharge of pollutants that could cause or contribute to violations of water quality standards.

While the high proportion of impervious area present on the large commercial site relative to pervious area would not allow eliminating all discharge, harvesting roof water and draining NCIA to properly-prepared pervious area would substantially decrease the volume discharged. Deployment of treatment control BMPs (e.g. sand filter treatment) could cut contaminant discharges from pollutants in the remaining volume of runoff to low levels.

Table 10 presents the pollutant reductions from the untreated case achievable through the complete LID approach described above in comparison to conventional treatments (from Table 6). Assuming EIA still discharges through sand filters, pollutant loadings from the untreated condition are expected to decrease by more than 96 percent for all but the COMM case. In that challenging case loadings would still fall by at least 89 percent for TSS and the metals and by 83 percent for total phosphorus, assuming City of Ventura rainfall levels, and slightly less assuming the higher Ojai rainfall levels. Thus, the Draft Permit's basic premise of disconnecting most impervious area, supplemented by specially managing roof water, is shown by both water quality and hydrologic results to be feasible and to afford broad and significant environmental benefits.

Table 10. Pollutant Loading Reduction Estimates With a Full LID Approach Relative to Conventional BMPs

	MFR[a]	Sm-SFR[a]	REST[a]	OFF[a]	Lg-SFR[a]	COMM[a]
Conventional TSS loading reduction[b]	15.7-69.9%	19.9-75.4%	22.0-80.6%	24.0-82.6%	19.9-75.4%	16.9-72.3%
Conventional TCu loading reduction[b]	0.0-74.4%	0.0-69.1%	0.0-78.2%	0.0-75.4%	0.0-69.1%	0.0-78.7%
Conventional TZn loading reduction[b]	22.7-78.4%	22.4-78.1%	22.9-84.3%	23.1-78.8%	22.4-78.1%	25.1-80.9%
Conventional TP loading reduction[b]	30.6-66.3%	41.5-70.7%	40.7-69.1%	45.9-76.2%	41.5-70.7%	20.3-55.0%
LID TSS loading reduction[c]	99.4%	99.3%	99.5%	99.4%	99.3%	89.0% [d]
LID TCu loading reduction[c]	98.1%	96.7%	98.0%	96.2%	96.7%	90.6% [d]
LID TZn loading reduction[c]	99.1%	98.8%	98.9%	98.3%	98.8%	94.8% [d]
LID TP loading reduction[c]	98.1%	98.6%	98.8%	98.7%	98.6%	83.1%[d]

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential; REST—restaurant; OFF—office building; Lg-SFR—large-scale single-family residential; COMM—retail commercial; CDS— continuous deflective separation unit; EDB—extended-detention basin; NCIA—not connected impervious area; EIA—effective (connected) impervious area

[b] Range from Table 6 represented by treatment by CDS unit, EDB, biofiltration swale, or biofiltration strip

[c] Based on directing roof runoff to downspout infiltration trenches (MFR, Sm-SFR, REST, and Lg-SFR) or harvesting it (OFF and COMM), draining other NCIA to pervious areas, and treating EIA with sand filters

[d] If the higher rainfall at Ojai is assumed, reduction estimates for TSS, TCu, TZn, and TP would be 84.0, 86.3, 92.5, and 75.5 percent, respectively.

SUMMARY AND CONCLUSIONS

This paper demonstrated that common Ventura County area residential and commercial development types subject to the Municipal NPDES Permit are likely, without storm water management, to reduce groundwater recharge from the predevelopment state by approximately half in most cases to a much higher fraction with a large ratio of impervious to pervious area. With no treatment, runoff from these developments is expected to exceed CTR acute copper and zinc criteria at the point of discharge and to deliver large pollutant mass loadings to receiving waters.

Conventional soil-based BMP solutions that promote and are component parts of low-impact development approaches, by contrast, regain about 30-50 percent of the recharge lost in development without storm water management, although commercially-manufactured filtration and hydrodynamic BMPs for storm water management give no benefits in this area. It is expected the soil-based BMPs generally would release effluent that meets the acute zinc criterion at the point of discharge, although it would still exceed the copper limit. Excepting phosphorus, it was found that these BMPs would capture and prevent the movement to receiving waters of the majority of the pollutant loadings considered in the analysis.

It was found that a three percent Effective Impervious Area standard can be met in typical developments, and that by draining all site runoff to pervious areas, runoff can be eliminated entirely in most development types. This result was reached assuming the use of native soils. Soil enhancement (typically, with compost) can further advance infiltration. Draining impervious surfaces onto the loam soils typical of Ventura County, in connection with limiting directly connected impervious area to three percent of the site total area, should eliminate storm runoff from some development types and greatly reduce it from more highly impervious types. Adding roof runoff elimination to the LID approach (by harvesting or directing it to downspout infiltration trenches) should eliminate runoff from all but mostly impervious developments. Even in the development scenario involving the highest relative proportion of impervious surface, losses of rainfall capture for beneficial uses could be reduced from more than 85 to less than 40 percent, and pollutant mass loadings would fall by 83-95 percent from the untreated scenario when draining to pervious areas was supplemented with water harvesting. These results demonstrate the basic soundness of the Draft Permit's concept to limit directly connected impervious area and drain the remainder over pervious surfaces.

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ATTACHMENT A

JUSTIFICATION OF PROPOSED EFFECTIVE IMPERVIOUS AREA LIMITATION

Summary

The literature shows that adverse impacts to the physical habitat and biological integrity of receiving waters occur as a result of the conversion of natural areas to impervious cover. These effects are observed at the lowest levels of impervious cover in associated catchments (two to three percent) and are pronounced by the point that impervious cover reaches five percent. To protect biological productivity, physical habitat, and other beneficial uses, effective impervious area should be capped at no more than three percent.

I. Impacts to physical habitat of California receiving waters observed at three percent impervious cover

Stein *et al*.[7] note that while studies from parts of the country with climates more humid than California's indicate that physical degradation of stream channels can initially be detected when watershed impervious cover approaches 10%, biological effects, which may be more difficult to detect, may occur at lower levels (CWP 2003).[8] Recent studies from both northern and southern California indicate that intermittent and ephemeral streams in California are more susceptible to the effects of hydromodification than streams from other regions of the US, with stream degradation being recognized when the associated catchment's impervious cover is as little as 3-5% (Coleman *et al*. 2005).[9] Furthermore, supplemental landscape irrigation in semi-arid regions, like California, can substantially increase the frequency of erosive flows (AQUA TERRA Consultants 2004).[10]

Coleman, *et al*.[3] report that the ephemeral/intermittent streams in southern California (northwestern Los Angeles County through southern Ventura County to central Orange County) appear to be more sensitive to changes in percent impervious cover than streams in other areas. Stream channel response can be represented using an *enlargement curve*, which relates the percent of impervious cover to a change in cross-sectional area. The data for southern California streams forms a relationship very similar in shape to the enlargement curves developed for other North American streams. However, the curve for southern California streams is above the general curve for streams in other climates. This suggests that a specific enlargement ratio is produced at a lower value of impervious surface area in southern California than in other parts of North America. Specifically, the estimated threshold of response is approximately 2-3% impervious cover, as compared to 7-10% for other portions of the U.S. It is important to note that this conclusion applies specifically to streams with a catchment drainage area less than 5 square miles.

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[7] Stein, E.D., S. Zaleski, (2005) *Managing Runoff to Protect Natural Streams: The Latest Developments on Investigation and Management of Hydromodification in California*. (Proceedings of a Special Technical Workshop Co-sponsored by California Stormwater Quality Association (CASQA), Stormwater Monitoring Coalition (SMC), University of Southern California Sea Grant (USC Sea Grant), Technical Report #475).

[8] Center for Watershed Protection (CWP), (2003) *Impacts of Impervious Cover on Aquatic Systems*. Ellicott City, MD.

[9] Coleman, D., C. MacRae, and E.D. Stein, (2005) *Effect of Increases in Peak Flows and Imperviousness on the Morphology of Southern California Streams.* Southern California Coastal Water Research Project Technical Report #450, Westminster, CA.

[10] AQUA TERRA Consultants, (2004) *Urbanization and Channel Stability Assessment in the Arroyo Simi Watershed of Ventura County CA*. FINAL REPORT. Prepared for Ventura County Watershed Protection Division, Ventura CA.

This study concludes that disconnecting impervious areas from the drainage network and adjacent impervious areas is a key approach to protecting channel stability. Utilizing this strategy can make it practical to keep the effective impervious cover (*i.e*. the amount hydrologically connected to the stream) equal to or less than the identified threshold of 2-3%.

II. Impacts to biological integrity of receiving waters observed with any conversion from natural to impervious surface

Two separate studies conducted by Horner *et al*.[11,12] in the Puget Sound region (Washington State), Montgomery County, Maryland, and Austin, Texas built a database totaling more than 650 reaches on low-order streams in watersheds ranging from no urbanization and relatively little human influence (the reference state, representing "best attainable" conditions) to highly urban (>60 percent total impervious area, "TIA"). Biological health was assessed according to the benthic index of biotic integrity (B-IBI) and, in Puget Sound, the ratio of young-of-the-year coho salmon (*Oncorhynchus kisutch*), a relatively stress-intolerant fish, to cutthroat trout (*Oncorhynchus clarki*), a more stress-tolerant species. The following discussion summarizes the results and conclusions of these two studies.

There is no single cause for the decline of water resource conditions in urbanizing watersheds. Instead, it is the cumulative effects of multiple stressors that are responsible for degraded aquatic habitat and water quality. Imperviousness, while not a perfect yardstick, appears to be a useful predictor of ecological condition. However, a range of stream conditions can be associated with any given level of imperviousness. In general, only streams that retain a significant proportion of their natural vegetative land-cover <u>and</u> have very low levels of watershed imperviousness appear to retain their natural ecological integrity. It is this change in watershed land-cover that is largely responsible for the shift in hydrologic regime from a sub-surface flow dominated system to one dominated by surface runoff.

While the decline in ecological integrity is relatively continuous and is consistent for all parameters, the impact on physical conditions appears to be more pronounced earlier in the urbanization process than chemical degradation. It is generally acknowledged, based on field research and hydrologic modeling, that it is the shift in hydrologic conditions that is the driving force behind physical changes in urban stream-wetland ecosystems.

Multiple scales of impact operate within urbanizing watersheds: landscape-level impacts, including the loss of natural forest cover and the increase in impervious surface area throughout the watershed; riparian corridor-specific impacts such as encroachment, fragmentation, and loss of native vegetation; and local impacts such as water diversions, exotic vegetation, stream channelization, streambank hardening, culvert installation, and pollution from the widespread use of pesticides and herbicides. All of these stressors contribute to the overall cumulative impact.

The researchers found that there is no clear threshold of urbanization below which there exists a "no-effect" condition. Instead, there appears to be a relatively continuous decline in almost all measures of water quality or ecological integrity. Losses of integrity occur from the lowest levels of TIA and are already pronounced by the point that TIA reaches 5 percent.

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[11] Horner, R. R., C. W. May, (2002) *The Limitations of Mitigation-Based Stormwater Management in the Pacific Northwest and the Potential of a Conservation Strategy based on Low-Impact Development Principles*. (Proceedings of the American Society of Engineers Stormwater Conference, Portland, OR).
[12] Horner, R.R., E. H. Livingston, C. W. May, J. Maxted, (2006) *BMPs, Impervious Cover, and Biological Integrity of Small Streams*. (Proceedings of the Eighth Biennial Stormwater Research and Watershed Management Conference, Tampa, FL).

Similarly, the Alliance for the Chesapeake Bay[13] reports that small-watershed studies by the Maryland Department of Natural Resources Biological Stream Survey have shown that some sensitive species are affected by even low amounts of impervious cover. In one study, no brook trout were observed in any stream whose watershed had more than 2 percent impervious cover, and brook trout were rare in any watershed with more than 0.5 percent impervious cover.

III. Ventura County's watersheds include biologically-significant water bodies

The literature discussed above is relevant to the watersheds of Ventura County, which contain rivers and streams that currently or historically support a variety of beneficial uses that may be impaired by water quality degradation and stream hydromodification as a result of storm water runoff from impervious land cover. Unlike some Southern California watersheds, Ventura County still has many natural stream systems with a high degree of natural functionality.

For instance, the Ventura River watershed in northwestern Ventura County "supports a large number of sensitive aquatic species,"[14] including steelhead trout, a federally-listed endangered species. Although "local populations of steelhead and rainbow trout have nearly been eliminated along the Ventura River" itself, the California Department of Fish and Game has "recognized the potential for the restoration of the estuary and enhancement of steelhead populations in the Ventura River."[15] Steelhead may also be present in tributaries such as San Antonio Creek.[16] Thriving rainbow trout populations exist in tributaries of the Ventura River including Matilija Creek and Coyote Creek.[17] The Ventura River either does or is projected to support the following beneficial uses: warm freshwater habitat; cold freshwater habitat; wildlife habitat; rare, threatened, or endangered species; migration of aquatic organisms; and spawning and reproduction.[18] Furthermore, the Ventura River Estuary also supports commercial fishing, shellfish harvesting, and wetland habitat.[19] The Ventura River receives municipal storm drain discharges from Ojai, San Buenaventura, and unincorporated areas of Ventura County.[20]

The Santa Clara River watershed in northern Ventura County "is the largest river system in southern California that remains in a relatively natural state."[21] Sespe Creek is one of the Santa Clara's largest tributaries, and "supports significant steelhead spawning and rearing habitat."[22] Other creeks in the Santa Clara River watershed that support steelhead are Piru Creek and Santa Paula Creek. Sespe Creek and the Santa Clara River also provide spawning habitat for the Pacific lamprey. Rainbow trout populations exist in tributaries of the Santa Clara River including Sespe Creek.[23] The creeks and the Santa Clara river do or are projected to support the following beneficial uses: warm freshwater habitat; cold freshwater habitat; wildlife habitat; preservation of biological habitats rare, threatened, or endangered species; migration of aquatic organisms; and spawning and reproduction.[24] Los Padres National Forest covers much of the Santa Clara River watershed, but increasing development in floodplain areas has been

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[13] Karl Blankenship, BAY JOURNAL,"It's a hard road ahead for meeting new sprawl goal: States will try to control growth of impervious" (July/August 2004), at http://www.bayjournal.com/article.cfm?article=66.
[14] Los Angeles Region Water Quality Control Plan (1994) p. 1-18 ("Basin Plan").
[15] Basin Plan, p. 1-16; Ventura County Environmental & Energy Resources Division, "Endangered Steelhead Trout in Ventura County: Past, Present, and Future," available at http://www.wasteless.org/Eye_articles/steelhead.htm.
[16] Ventura County Environmental & Energy Resources Division, "Steelhead Spawning in Ventura County," (2005), available at http://www.wasteless.org/Eye_articles/steehead2005.html.
[17] Ventura County Environmental & Energy Resources Division, "Endangered Steelhead Trout in Ventura County: Past, Present, and Future," available at http://www.wasteless.org/Eye_articles/steelhead.htm.
[18] Basin Plan, Table 2-1.
[19] Basin Plan, Table 2-4.
[20] Ventura County Watershed Protection District, *Report of Waste Discharge* (January 2005) at p. 3.
[21] Basin Plan, p. 1-16.
[22] Basin Plan, p. 1-16.
[23] Ventura County Environmental & Energy Resources Division, "Endangered Steelhead Trout in Ventura County: Past, Present, and Future," available at http://www.wasteless.org/Eye_articles/steelhead.htm.
[24] Basin Plan, Table 2-1.

identified as a threat to the river system's water quality.[25] Furthermore, the Santa Clara estuary supports the additional beneficial uses of shellfish harvesting and wetlands habitat.[26] The Santa Clara River receives municipal storm drain discharges from Fillmore, Oxnard, San Buenaventura, Santa Paula, and unincorporated areas of Ventura County.[27]

The Calleguas Creek watershed "empties into Mugu Lagoon, one of southern California's few remaining large wetlands."[28] It supports or is projected to support the following beneficial uses: estuarine habitat; marine habitat; wildlife habitat; preservation of biological habitats; rare, threatened, or endangered species; migration of aquatic organisms; spawning and reproduction; shellfish harvesting; and wetlands habitat.[29] Historically, Calleguas Creek drained largely agricultural areas. But this watershed has been under increasing pressure from sedimentation due to increased surface flow from municipal discharges and urban wastewaters, among other sources.[30] Increasing residential developments on steep slopes has been identified as a substantial contributing factor to the problem of accelerated erosion in the watershed (and sedimentation in the Lagoon). Calleguas Creek receives municipal storm drain discharges from Camarillo, Moorpark, Simi Valley, Thousand Oaks, and unincorporated areas of Ventura County.[31]

Ventura County's coastal streams also support a variety of beneficial uses:[32]

- Little Sycamore Canyon Creek in southern Ventura County (warm freshwater habitat; wildlife habitat; rare, threatened or endangered species; and spawning and reproduction);
- Lake Casitas tributaries (warm freshwater habitat; cold freshwater habitat; wildlife habitat; rare, threatened or endangered species; spawning and reproduction; and wetland habitat);
- Javon Canyon and Padre Juan Canyon (warm freshwater habitat; cold freshwater habitat; wildlife habitat; and spawning and reproduction); and
- Los Sauces Creek in northern Ventura County (warm freshwater habitat; cold freshwater habitat; wildlife habitat; migration of aquatic species; and spawning and reproduction).

IV. Conclusion

In order to protect the biological habitat, physical integrity, and other beneficial uses of the water bodies in Ventura County, effective impervious area should be capped at no more than three percent.

[25] Basin Plan, pp. 1-16, 1-18.
[26] Basin Plan, Table 2-4.
[27] Ventura County Watershed Protection District, *Report of Waste Discharge* (January 2005) at p. 3.
[28] Basin Plan, p. 1-18.
[29] Basin Plan, Table 2-1.
[30] Basin Plan, pp. 1-16, 1-18.
[31] Ventura County Watershed Protection District, *Report of Waste Discharge* (January 2005) at p. 3.
[32] Basin Plan, Table 2-1.

ATTACHMENT B

POLLUTANT CONCENTRATIONS FOR URBAN SOURCE AREAS (HERRERA ENVIRONMENTAL CONSULTANTS, INC. UNDATED)

Source Area	Study	Location	Sample Size (n)	TSS (mg/L)	TCu (ug/L)	TPb (ug/L)	TZn (ug/L)	TP (mg/L)	Notes
Roofs									
Residential	Steuer, et al. 1997	MI	12	36	7	25	201	0.06	2
Residential	Bannerman, et al. 1993	WI	~48	27	15	21	149	0.15	3
Residential	Waschbusch, et al. 2000	WI	25	15	n.a.	n.a.	n.a.	0.07	3
Residential	FAR 2003	NY		*19*	*20*	*21*	*312*	*0.11*	4
Residential	Gromaire, et al. 2001	France		*29*	*37*	*493*	*3422*	*n.a.*	5
Representative Residential Roof Values				**25**	**13**	**22**	**159**	**0.11**	
Commercial	Steuer, et al. 1997	MI	12	24	20	48	215	0.09	2
Commercial	Bannerman, et al. 1993	WI	~16	15	9	9	330	0.20	3
Commercial	Waschbusch, et al. 2000	WI	25	18	n.a.	n.a.	n.a.	0.13	3
Representative Commercial Roof Values				**18**	**14**	**26**	**281**	**0.14**	
Parking Areas									
Res. Driveways	Steuer, et al. 1997	MI	12	157	34	52	148	0.35	2
Res. Driveways	Bannerman, et al. 1993	WI	~32	173	17	17	107	1.16	3
Res. Driveways	Waschbusch, et al. 2000	WI	25	34	n.a.	n.a.	n.a.	0.18	3
Driveway	FAR 2003	NY		*173*	*17*		*107*	*0.56*	4
Representative Residential Driveway Values				**120**	**22**	**27**	**118**	**0.66**	
Comm./ Inst. Park. Areas	Pitt, et al. 1995	AL	16	110	116	46	110	n.a.	1
Comm. Park. Areas	Steuer, et al. 1997	MI	12	110	22	40	178	0.2	2
Com. Park. Lot	Bannerman, et al. 1993	WI	5	58	15	22	178	0.19	3
Parking Lot	Waschbusch, et al. 2000	WI	25	51	n.a.	n.a.	n.a.	0.1	3
Parking Lot	Tiefenthaler, et al. 2001	CA	5	36	28	45	293	n.a.	6
Loading Docks	Pitt, et al. 1995	AL	3	40	22	55	55	n.a.	1
Highway Rest Areas	CalTrans 2003	CA	53	*63*	*16*	*8*	*142*	*0.47*	7
Park and Ride Facilities	CalTrans 2003	CA	179	*69*	*17*	*10*	*154*	*0.33*	7
Comm./ Res. Parking	FAR 2003	NY		*27*	*51*	*28*	*139*	*0.15*	4
Representative Parking Area/Lot Values				**75**	**36**	**26**	**97**	**0.14**	

Landscaping/Lawns									
Landscaped Areas	Pitt, et al. 1995	AL	6	33	81	24	230	n.a.	1
Landscaping	FAR 2003	NY		*37*	*94*	*29*	*263*	n.a.	4
Representative Landscaping Values				**33**	**81**	**24**	**230**	**n.a.**	
Lawns - Residential	Steuer, et al. 1997	MI	12	262	n.a.	n.a.	n.a.	2.33	2
Lawns - Residential	Bannerman, et al. 1993	WI	~30	397	13	n.a.	59	2.67	3
Lawns	Waschbusch, et al. 2000	WI	25	59	n.a.	n.a.	n.a.	0.79	3
Lawns	Waschbusch, et al. 2000	WI	25	122	n.a.	n.a.	n.a.	1.61	3
Lawns - Fertilized	USGS 2002	WI	58	n.a.	n.a.	n.a.	n.a.	2.57	3
Lawns - Non-P Fertilized	USGS 2002	WI	38	n.a.	n.a.	n.a.	n.a.	1.89	3
Lawns - Unfertilized	USGS 2002	WI	19	n.a.	n.a.	n.a.	n.a.	1.73	3
Lawns	FAR 2003	NY	3	*602*	*17*	*17*	*50*	*2.1*	4
Representative Lawn Values				**213**	**13**	**n.a.**	**59**	**2.04**	

Notes:

Representative values are weighted means of collected data. Italicized values were omitted from these calculations.

1 - Grab samples from residential, commercial/institutional, and industrial rooftops. Values represent mean of DETECTED concentrations

2 - Flow-weighted composite samples, geometric mean concentrations

3 - Geometric mean concentrations

4 - Citation appears to be erroneous - original source of data is unknown. Not used to calculate representative value

5 - Median concentrations. Not used to calculate representative values due to site location and variation from other values.

6 - Mean concentrations from simulated rainfall study

7 - Mean concentrations. Not used to calculate representative values due to transportation nature of land use.

INVESTIGATION OF THE FEASIBILITY AND BENEFITS OF LOW-IMPACT SITE DESIGN PRACTICES APPLIED TO MEET VARIOUS POTENTIAL STORMWATER RUNOFF REGULATORY STANDARDS

By

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Report to

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EXECUTIVE SUMMARY

STUDY DESIGN

A study was performed to investigate the degree to which stormwater management practices, commonly referred to as “low-impact development” methods or “green infrastructure,” can retain urban runoff and meet five possible regulatory standards that could be applied nationally. Retention is defined as preventing the conversion of precipitation to runoff discharging from a development site on the surface, from where it can enter a receiving water. Retaining runoff from impervious and pollutant generating pervious surfaces prevents the introduction of urban runoff pollutants to receiving waters as well as reduces runoff volume to prevent stream channel and habitat damage, flooding, and loss of groundwater recharge. ARCD methods were assessed for their ability to: (1-2) meet standards pertaining to retention of the runoff generated by the 85th and 95th percentile, 24-hour precipitation events; (3) retain 90 percent of the post-development runoff; and (4-5) retain the difference between the post- and pre-development runoff, both with and without a cap at the 85th percentile, 24-hour event. The study assessed five urban land use types (three residential, one retail commercial, and one infill redevelopment), each placed in four climate regions in the continental United States on two regionally common soil types.

Infiltrating bioretention was applied as an initial strategy in the analysis of each case. When the initial strategy could not fully retain post-development runoff, additional methods were applied, involving roof runoff harvesting in the most impervious development cases and roof water dispersion in those with substantial pervious area. Benefits were assessed with respect to reduction of the annual average surface runoff volume from the quantity estimated without any stormwater management practices, the associated maintenance of pre-development groundwater recharge, and water quality improvement achieved through preventing discharge to receiving waters of pollutants generated with developed land uses.

RETENTION AND POLLUTANT REDUCTION CAPABILITIES

The initial strategy of infiltrating bioretention could retain all post-development runoff and pre-existing groundwater recharge, as well as attenuate all pollutant transport, in the three residential land use development types on hydrologic soil group (HSG) B soils, in all cases, in all regions, taking a fraction of the available pervious area to do so. For the more highly impervious commercial retail and redevelopment cases, bioretention would retain about 45 percent of the runoff and pollutants generated and save about 40 percent of the pre-development recharge. Adding roof runoff management measures in these cases would approximately double retention and pollutant reduction for the retail commercial land use and raise it to 100 percent for the redevelopment. Results were generally similar with HSG C soils, although more of the pervious portion of sites was required to equal the retention seen on B soils.

For development on the D soils in all climate regions, use of roof runoff management techniques was estimated to increase runoff retention and pollutant reduction from zero to between about one-third to two-thirds of the post-development runoff generated, depending on the land use case. These strategies would offer little groundwater recharge benefit with this soil condition, but would still have the potential to significantly reduce runoff volume and pollutant loading.

ABILITY TO MEET STANDARDS

The projected ability to meet the five standards identified above was found to vary mostly in relation to soil type (B or C versus D) and the relative imperviousness of development. The ability to meet the five standards varied much less across climate regions. With B and C soils, the methods considered were projected to meet all five standards in all but 12 of 125 evaluations. With D soils, however, only three standards could be met at all and those only occasionally. However, even on D soils, all cases for Standard 1 (retention of the 85^{th} percentile, 24-hour precipitation event) were able to retain greater than 50 percent of the required runoff volume. Moreover, opportunities to use ARCD practices or site design principles not modeled in this analysis have the potential to further increase runoff retention volume.

Standard 3 (retain 90 percent of the average annual post-development runoff volume) would be the most environmentally protective standard. Meeting or coming as close as possible to meeting, but not exceeding, this standard was estimated to lead to 66-90 percent of total runoff retention and pollutant loading reduction on B and C soils and 37-66 percent runoff retention on D soils. Standard 2 (retain the runoff produced by the 95^{th} percentile, 24-hour precipitation event) would yield equivalent protection on D soils and only slightly less protection with B and C soils. The outcome with this standard would also be more consistent region to region than with the alternative standard 1, based on the 85^{th} instead of the 95^{th} percentile precipitation event. Sites located on B or C soils were able retain the runoff produced by the 85^{th} percentile storm in 24 of 25 cases modeled (in 18 of the 25 cases by using infiltrating bioretention alone), and were able to retain the runoff produced by the 95^{th} percentile storm in 22 of 25 cases modeled.

Standards 4 and 5, based on the differential between pre- and post-development runoff volume, are inconsistent in retaining runoff and reducing pollutants, in that they are relatively protective where pre-development runoff is estimated to be low relative to post-development flow, but result in progressively lower retention and pollutant loading reduction as pre- and post-development volumes converge, such as in several cases on D soils. Standard 5 is especially weak in this regard. The potentially low level of retention and pollutant loading reduction renders these standards based on the change in pre- versus post-development runoff volume poor candidates for national application, at least as formulated in these terms.

In summary, standards 2 and 3 are clearly superior to the other three options from both a volume and pollutant load reduction standpoint. Standard 3 is entirely consistent from place to place in degree of environmental protection, and standard 2 does not deviate much. Analysis of the five development cases on two soil groups in each of four regions demonstrated the two standards are virtually identical in the runoff retention and pollutant loading reduction they would bring about. Of the remaining standards, standard 1 (retantion of the runoff produced by the 85^{th} percentile storm event) remains more consistent across regions and more protective of water quality for development on D soils than either standard 4 or 5, and is preferable to those standards in this regard.

INTRODUCTION

GENERAL STUDY DESCRIPTION

Study Design

This purpose of this study was to investigate the degree to which low-impact development (LID) [1] practices can meet or exceed the requirements of various potential stormwater management facility design standards and to determine the environmental benefits that can be realized by applying these techniques. The investigation was performed by estimating the stormwater retention possible with full application of low-impact options under a range of conditions broadly representative of different regions within the United States and then determining the implications of the findings for achieving various standards and for providing benefits. Retention is defined as preventing the conversion of precipitation to surface runoff from urbanized land uses through infiltration, evapotranspiration, and/or harvesting for some water supply purpose. Retaining runoff from impervious and pollutant generating pervious surfaces prevents the introduction of urban runoff pollutants to receiving waters as well as reduces runoff volume to prevent stream channel and habitat damage, flooding, and loss of groundwater recharge. Benefits were assessed with respect to reduction of the potential developed land surface runoff volume, the associated maintenance of pre-development groundwater recharge, and water quality improvement achieved through preventing discharge to receiving waters of pollutants generated with developed land uses.

The potential regulatory standards investigated were capture and retention of, at minimum:

- Standard 1—The runoff produced by the 85th percentile, 24-hour precipitation event,[2] a standard commonly used in California;
- Standard 2—The runoff produced by the 95th percentile, 24-hour precipitation event, the standard adopted under Section 438 of the Energy Independence and Security Act;
- Standard 3—90 percent of the average annual post-development runoff volume;
- Standard 4—The difference between the post- and pre-development[3] average annual runoff volumes; and
- Standard 5—The difference between the post- and pre-development runoff volumes for all events up to and including the 85th percentile, 24-hour precipitation event.

Conditions broadly representative of the nation were selected by, first, considering the climate regions defined in USEPA's (1983) Nationwide Urban Runoff Project (NURP) report. For full analysis, climate regions 1 (Northeast-Upper Midwest), 3 (Southeast), 5 (South Central), and 6 (Southwest) were chosen as providing a wide range of climatological conditions and geographic distribution. Once the four regions were picked, a metropolitan area and a specific city in each were chosen to serve as typical models of development circumstances in the general area, as

[1] The National Research Council (NRC, 2009) renamed LID, also known as green infrastructure, as aquatic resources conservation design (ARCD), the term used henceforth in this report.

[2] The 85th percentile, 24-hour event represents the precipitation quantity in a 24-hour period not exceeded in 85 percent of all events in an extended record.

[3] In this study the pre-development state is taken as the typical land cover existing before European settlement of an area.

detailed in the Case Studies discussion below. In addition, region 7 (Pacific Northwest) was identified as an additional location to be discussed. This region is the site of a considerable amount of ARCD application in an area somewhat different climatologically than other selected regions, in having persistent winter rainfall totaling annually, in the major urban areas, intermediately among the other regions. Results of research on ARCD conducted in this region are discussed at several points in this report.

Soils and topography were the next considerations in developing broadly representative conditions. U.S. Department of Agriculture websites were the source of general soil characterizations for the study regions and specific soil survey data in and around the representative metropolitan areas. Soils generally represented some range in textural classes and associated hydraulic conductivities. For each region, a soil type predominating among those representing hydraulic conductivities relatively high and low for the region were selected to serve as a basis for the analyses. The effect of slope was also investigated but ultimately found not to affect results substantially.

Five types of urban development were selected to represent breadth in land use: (1) multi-family residential, (2) small-scale single-family residential, (3) large-scale single-family residential, (4) large-scale commercial, and (5) infill redevelopment. Building permit data from each region were consulted to determine typical distributions of site features for each (e.g., land cover by buildings, parking areas, roadways, walkways, driveways, landscaping).

Case studies thus comprised four climate regions, each with two soil conditions and five land use types, for a total of 40 permutations. For each, the ability of the site to accommodate soil- and vegetation-based ARCD practices was investigated. Runoff quantities were estimated and compared to the five potential regulatory standards. Annual mass loading discharges were estimated for four pollutants: total suspended solids (TSS), total recoverable copper (TCu) and zinc (TZn), and total phosphorus (TP). In any case where soil- and vegetation-based ARCD infiltration techniques appeared not to be able to attenuate all runoff, specific roof runoff management strategies were investigated as possible measures to achieve additional retention. Runoff quantities and pollutant discharges were recalculated based on use of these additional practices in place.

This report covers the methods employed in the investigation, data sources, and references for both. It then presents the results, discusses their consequences, draws conclusions, and makes recommendations relative to the feasibility of utilizing low-impact development practices to meet the respective potential regulatory standards.

AQUATIC RESOURCES CONSERVATION DESIGN PRACTICES

General Description

As the stormwater management field developed, it passed through several stages. First, it was thought that the key to success was to match post-development with pre-development peak flow rates, while also reducing a few common pollutants (usually, TSS) by a set percentage. Finding that these efforts generally required large ponds, but that they did not forestall impacts, stormwater managers next deduced that runoff volumes and high discharge durations would also have to decrease. Almost simultaneously, although not necessarily in concert, the idea of low-impact development arose to offer a way to achieve actual avoidance, or at least minimization, of discharge quantity and pollutant increases reaching far above pre-development levels. These methods reduce storm runoff and its contaminants by decreasing their generation at sources, infiltrating into the soil or evaporating or transpiring[4] storm flows before they can enter surface receiving waters, and treating flow remaining on the surface through contact with vegetation and soil, or a combination of these strategies.

The National Research Council ("NRC") (2009) renamed LID as Aquatic Resources Conservation Design (ARCD) for several reasons. First, this term signifies that the principles and many of the methods apply not only to building on previously undeveloped sites, but also to redeveloping and retrofitting existing development. Second, incorporating aquatic resources conservation in the title is a direct reminder of the central reason for improving stormwater regulation and management. ARCD encompasses the complete range of practices to counteract all negative urban runoff impacts; i.e., the full suite of practices that emphasize and accomplish retention as defined above. These practices aim at decreasing surface runoff peak flow rates, volumes, and elevated flow durations, as well as avoiding or at least minimizing the introduction of pollutants to any surface runoff produced. Reducing the concentration of pollutants, together with runoff volume decrease, cuts the cumulative mass loadings (mass per unit time) of pollutants entering receiving waters over time.

The menu of ARCD practices begins with conserving, as much as possible, existing trees, other vegetation, and soils, as well as natural drainage features (e.g., depressions, dispersed sheet flows, swales). Clustering development to affect less land is a fundamental practice advancing this goal. Conserving natural features would further entail performing construction in such a way that vegetation and soils are not needlessly disturbed and soils are not compacted by heavy equipment. Using less of polluting materials, isolating contaminating materials and activities from contact with rainfall or runoff, and reducing the introduction of irrigation and other non-stormwater flows into storm drain systems are essential. Many ARCD practices fall into the category of minimizing impervious areas through decreasing building footprints and restricting the widths of streets and other pavements to the minimums necessary. Another important category of ARCD practices involves directing runoff from roofs and pavements onto pervious areas as sheet flow, where all or much of the runoff can infiltrate or evaporate in many situations.

Water can be harvested from impervious surfaces, especially roofs, and put to use for irrigation, non-potable indoor water supply. Harvesting is a standard technique for Leadership in Energy and Environmental Design (LEED) buildings (U.S. Green Building Council, 2008). Many successful systems of this type are in operation, with examples such as the Natural Resources Defense Council offices (Santa Monica, CA), the King County Administration Building (Seattle, WA), and two buildings on the Portland State University campus (Portland, OR). Harvesting is feasible at the small scale using rain barrels and at larger scales using larger collection cisterns and piping systems. These small-scale applications have been used throughout the world for centuries and are rapidly spreading in the United States today (See, e.g., Texas Water Development Board, 2005; Georgia Department of Community Affairs, 2009).

If these practices are used but runoff is still produced, ARCD offers an array of techniques to retain it on-site through infiltration and evapotranspiration (ET). The bioretention cell (rain garden) is the workhorse practice in this category, but swales conveying flow slowly, filter strips set up for sheet flows, and other modes are also important. Relatively low traffic areas can be constructed with permeable surfaces such as porous asphalt, open-graded Portland cement concrete, coarse granular materials, concrete or plastic unit pavers, or plastic grid systems to allow for infiltration.

[4] Transpiration refers to vaporization of water from plant tissue, while evaporation applies to vaporization from a liquid (e.g., pool) or solid (e.g., leaf) surface. The terms are often combined to form the compound evapotranspiration (ET).

ARCD practices should be selected and applied as close to sources as possible to stem runoff and pollutant production near the point of potential generation. However, these practices must also work well together and, in many cases, must be supplemented with strategies operating farther downstream. For example, the City of Seattle, in its "natural drainage system" retrofit initiative, built serial bioretention cells flanking relatively flat streets. "Cascades" of vegetated stepped pools created by weirs were installed along more sloping streets. In some cases the cells drain to downstream cascades. The upstream components are highly effective in attenuating most or even all runoff. Flowing at higher velocities on sloped surfaces, the cascades do not perform at such a high level, although under favorable conditions they can still infiltrate or evapotranspire the majority of the incoming runoff (Chapman 2006, Chapman and Horner 2010). Even if not as impressive statistically, cascades can actually decrease storm discharge to streams more than the cells do, because of their generally greater size. Also, the cascades extract pollutants from remnant runoff through mechanisms mediated by vegetation and soils. The success of Seattle's natural drainage systems demonstrates that well designed ARCD practices can mimic natural landscapes hydrologically, and thereby avoid raising discharge quantities.

A watershed-based program emphasizing ARCD practices would convey significant benefits beyond greatly improved stormwater management. ARCD techniques overall would advance water conservation, and infiltrative practices would increase recharge of groundwater resources. ARCD practices can be made attractive and thereby improve neighborhood aesthetics and property values. Retention of more natural vegetation can both save wildlife habitat and provide recreational opportunities. Municipalities could use the program in their general urban improvement initiatives, giving incentives to property owners to contribute to goals in that area while also protecting water resources.

A Catalogue of ARCD Practices

ARCD practices are numerous and expanding as existing configurations are applied in new ways. Table 1 presents a catalogue adapted from USEPA (2007) and NRC (2009). This catalogue contains practices that are not equally applicable in all settings; e.g., nevertheless, each category offers practices applicable in a broad variety of circumstances.

The best strategy for choosing among and implementing these practices is a decentralized, integrated one; i.e., selecting practices that fit together as a system, starting at or near sources and working through the landscape until management objectives are met. This strategy makes maximum possible use of practices in the first three categories, which prevent stormwater quantity and quality problems, and then selects among the remaining classifications in relation to the localized and overall site conditions. Source control and preservation of existing vegetation and soils obviously avoid post-development runoff quantity and pollutant increases from any portion of the site that can be so treated. Among all strategies, these best maintain natural infiltration and ET patterns and yield of materials flowing from the site. This preventive strategy is supplemented by strategies to create as little impervious cover as possible. The remaining practices then contend with the excess runoff and pollutants over pre-development levels generated by the development.

For the practices that infiltrate water, a site's soil characteristics and depth to groundwater can and should be determined through infiltration rate testing and excavation to determine the infiltration capability. Because of the often substantial variability of conditions around a site, these determinations should be made at multiple points. If the natural infiltration rate is low, generally < 0.5 inch/hour (< 1.25 cm/h, Geosyntec 2008), in many situations the soil can be amended, usually with organic compost, to apply an infiltrative practice.

In addition to soil characteristics, the position of the groundwater table is a crucial determinant of whether or not stormwater infiltration should be promoted by applying ground-based ARCD practices. A seasonal high water table too close to the surface results in rapid saturation of a thin soil column and retarded infiltration. Ponding water longer than 72 hours can permit mosquito growth, damage vegetation, and promote clogging of the facility by microorganism growths and polysaccharide organic materials that form in the reduced-oxygen environment accompanying excessive ponding time (Mitchell and Nevo 1964, Ronner and Wong 1996). Also, storm runoff flow through a short soil column or very rapidly through a coarse-textured soil can convey contaminants to groundwater.

Evidence gathering from available performance data is that evapotranspiration (ET) can be a substantial factor in water retention (discussed below) but may be difficult to quantify at a given site without more research. A conservative approach is to design on the basis of infiltration rate, calculated to include consideration of soil amendments, if any. Together with careful investigation of soils and hydrogeologic conditions, this means of proceeding is very likely to produce facilities that retain at least as much runoff as predicted, and almost certainly more as a result of unquantified ET.

Table 1. A Catalogue of Aquatic Resources Conservation Design Practices (USEPA [2007] and NRC [2009])

Category	Definition	Examples
Source control	Minimizing pollutants or isolating them from contact with rainfall or runoff	• Substituting less for more polluting products • Segregating, covering, containing, and/or enclosing pollutant-generating materials, wastes, and activities • Avoiding or minimizing fertilizer and pesticide applications • Removing animal wastes deposited outdoors • Conserving water to reduce non-stormwater discharges
Conservation site design	Minimizing the generation of runoff by preserving open space and reducing the amount of land disturbance and impervious surface	• Clustering development • Preserving wetlands, riparian areas, forested tracts, and porous soils • Reducing pavement widths (streets, sidewalks, driveways, parking lot aisles) • Reducing building footprints
Conservation construction	Retaining vegetation and avoiding removing topsoil or compacting soil	• Minimizing site clearing • Minimizing site grading • Prohibiting heavy vehicles from driving anywhere unnecessary
Runoff harvesting	Capturing rainwater, generally from roofs, for a beneficial use	• Using storage and distribution systems (rain barrels or cisterns) for irrigation and/or indoor supply for public and private buildings
Natural runoff conveyance practices	Maintaining natural drainage patterns (e.g., depressions, natural swales) as much as possible, and designing drainage paths to increase the time before runoff leaves the site	• Emphasizing sheet instead of concentrated flow • Eliminating curb-and-gutter systems in favor of natural drainage systems • Roughening land surfaces • Creating long flow paths over landscaped areas • When flow must be concentrated, using vegetated channels with flow controls (e.g., check dams)
Practices for temporary runoff storage followed by infiltration and/or evapotranspiration[a]	Use of soil pore space and vegetative tissue to increase the opportunity for runoff to percolate to groundwater or vaporize to the atmosphere	• Bioretention cells (rain garden) • Vegetated swales (channel flow) • Vegetated filter strips (sheet flow) • Planter boxes • Tree pits • Infiltration basins • Infiltration trenches • Roof downspout surface or subsurface dispersal • Permeable pavement • Vegetated (green) roofs
ARCD landscaping[b]	Soil amendment and/or plant selection to increase storage, infiltration, and evapotranspiration	• Organic compost soil amendments • Native, drought-tolerant plantings • Reforestation • Turf conversion to meadow, shrubs, and/or trees

[a] Some of these practices are also conventional stormwater BMPs but are ARCD practices when ARCD landscaping methods are employed as necessary to maximize storage, infiltration, and evapotranspiration. The first five examples can be constructed with an impermeable liner and an underdrain connection to a storm sewer, if full retention is technically infeasible (see further discussion later). Vegetated roofs store and evapotranspire water but offer no infiltration opportunity, unless their discharge is directed to a secondary, ground-based facility.

[b] Selection of landscaping methods depends on the ARCD practice to which it applies and the stormwater management objectives, but amending soils unless they are highly infiltrative and planting several vegetation canopy layers (e.g., herbaceous growth, shrubs, and trees) are generally conducive to increasing storage, infiltration, and evapotranspiration.

Application of ARCD Practices in This Study

The investigation performed for this study first assessed the capacity of each case study site to infiltrate the full average annual post-development storm runoff volume and thereby reduce pollutant releases to zero. The report terms this initial evaluation as the “Basic ARCD Analysis”. The means of infiltration was not distinguished at this level of analysis. For example, it was not specified if runoff would be distributed in sheet flow across a pervious area or channeled into a rain garden. As detailed later in the Methods of Analysis section, this analysis was limited to the estimated infiltration capacity of the case study soil type, possibly compost-amended, and the available pervious area.

Critically, there was no attempt to estimate the loss of surface runoff through ET in the Basic ARCD analysis (ET is considered, to address rooftop runoff only, as part of our “Full ARCD analysis,” discussed below). In general, the estimated mean annual evapotranspiration in the Southeast is about 70 percent of the precipitation, or roughly 35 inches per year. For large areas of the Southwest, evapotranspiration is virtually equal to 100 percent of the precipitation, which is only about 10 inches per year. The ratio of estimated mean annual evapotranspiration to precipitation is least in the mountains of the Pacific Northwest and New England where evapotranspiration is about 40 percent of the precipitation (Hanson, 1991). By leaving out these substantial losses, generally 40 percent of precipitation or more, the retention estimates in this study can be considered quite conservative.

Additionally, there was no consideration of many ARCD practices in the Table 1 catalogue that could be applied in site-specific design. For example, there were no refinements of the prevailing building standards to reduce street widths or cluster buildings and reduce their footprints. Further, green roofs were not considered in this study, although they are already making a contribution to runoff reduction around the nation and reflect a significant additional opportunity to retain runoff on-site. The U.S. EPA has stated that “a 3.5-4 in. (8 -10 cm) deep green roof can retain 50% or more of the annual precipitation.” (U.S. EPA, 2009a). For water quality, we did not assume any source control implementation. Thus, actual site design could take advantage of substantial additional capabilities not considered in this study.

In cases where the practices incorporated in the initial level of analysis (infiltration through bioretention) did not, according to the estimates, fully attenuate post-development pollutant discharges, specific attention was directed at ways of extracting additional water from surface discharge by managing roof runoff. This assessment is called the “Full ARCD Analysis” in the report. The options broadly divide into harvesting water for a purpose such as irrigation and/or non-potable indoor supply, or making special provisions to infiltrate or evapotranspire roof runoff even if soil conditions are limiting. Harvesting applies best to relatively large developments having sufficient demand for the collected water. While single-family residences can harvest water into rain barrels or cisterns for lawn and garden watering, these containers may be small in volume relative to runoff production; and though opportunity exists, no credit was taken for them in this study. However, even in poorly infiltrating soils, options exist to disperse house roof runoff as sheet flow for storage in vegetation and soil until evapotranspiration and some infiltration occurs.

CASE STUDIES

CLIMATE REGIONS

Basis of Selection

The Nationwide Urban Runoff Project divided the nation into nine regions based on differences in volume, intensity, and duration of precipitation and interval between precipitation events (USEPA 1983). For broad representation of the U.S. generally this study chose regions 1 (Northeast-Upper Midwest), 3 (Southeast), 5 (South Central), and 6 (Southwest) for analysis. Table 2 provides the annual precipitation statistics from the NURP compilation.

Table 2. Precipitation Statistics (Means) for Four NURP Regions Selected for Study (USEPA 1983)

Region	Volume (inch)	Intensity (inch/hour)	Duration (hours)	Interval (hours)
1—Northeast-Upper Midwest	0.26	0.051	5.8	73
3—Southeast	0.49	0.102	5.2	89
5—South Central	0.33	0.080	4.0	108
6—Southwest	0.17	0.045	3.6	277

The selected regions represent a volume differential of about a factor of three, intensity variation of approximately two times, and inter-storm interval varying by almost four times. The NURP report shows coefficients of variation (mean/standard deviation) of greater than 1.0 for all of these means, indicating an overall high degree of dispersion.

Figure 1 visually depicts variation in mean annual precipitation across the continental United States. It shows that the selected regions are overall representative of the broadly prevailing range across the nation, particularly its major urban and still urbanizing areas.

Region 7 (Pacific Northwest) was also identified for discussion of research results on ARCD, although not full analysis. It has less intense (mean 0.024 inch/hour) but much more extended (mean 20.0 hours) precipitation compared to any other region in the nation. Mean storm volume ranks with region 3 (mean 0.48 inch); but fewer storms, especially in the summer, yield overall less total annual precipitation in lowland areas holding all urban development in region 7. It was of interest because of the already occurring use of ARCD techniques in a relatively rainy part of the country.

Representative Metropolitan Areas and Cities

Once the regions were identified, a metropolitan area within each area was chosen as a basis for assigning specific precipitation and development characteristics. The areas considered were USEPA-designated Urban Areas: “An urbanized area is a land area comprising one or more places – central place(s) – and the adjacent densely settled surrounding area – urban fringe – that together have a residential population of at least 50,000 and an overall population density of at least 1,000 people per square mile” (USEPA 2007). Stormwater regulations would have the most impact in areas that are being quickly developed, redeveloped, or both. Five of the twenty fastest growing counties in the nation from 2000 to 2009 were near Atlanta, GA and five were in the state of Texas (U.S. Census Bureau 2010). These statistics factored into the decision to focus on records from these regions.

Each selected metropolitan area is generally representative of its region in precipitation and development characteristics. Each is also undergoing relatively active new development and redevelopment, offering candidate locations where a prospective stormwater standard would frequently be applied. These metropolitan areas are: region 1—Boston, MA, region 3—Atlanta, GA, region 5—Austin, TX, and region 6—San Diego, CA

Figure 1. Precipitation of the Conterminous States of the United States, National Atlas of the United States, 2011.

Finally, a city with a high rate of development (and often redevelopment) was picked in each metropolitan area for investigation of building patterns and standards. The intent was to match regional patterns of climate, soils (see discussion on physiographic data, below), and land use and land cover realistically. After substantial investigation, the conclusion was that building standards, how land is used, and the relative allocation of impervious and pervious lands do not vary in any systematic way across the nation and cannot be regionally distinguished. Therefore, the variables of interest came down to precipitation and soils.

Alpharetta, about 30 miles north of Atlanta, represents that metropolitan area. In 1981 it was a small town of approximately 3,000 residents but grew to 51,243 by 2007. During the workday, the city swells to more than 120,000 residents, workers, and visitors. Alpharetta is home to large corporations such as AT&T (3500 employees), Verizon Wireless (3000 employees), and ADP, Inc./National Account Services (2100 employees). Infill redevelopment projects are anticipated in the downtown area (City of Alpharetta, 2011).

Round Rock is a typical developing city located 15 miles to the north of Austin, TX. In 1970 there were only 2,700 residents in this town, while today the population exceeds 100,000. Round Rock is the eighth-fastest growing city in the nation and the location of several large corporate campuses.

The Town of Framingham, 20 miles west of Boston, represents the northeastern climate zone. At nearly 67,000 inhabitants, Framingham is the largest entity designated as a “town” in the Commonwealth of Massachusetts. It is home to three large corporations and overall 2200 businesses providing 45,000 jobs. Differing greatly from the representative communities in other regions, Framingham was incorporated in 1700 and developed early in the nation's history. Today's activity includes redevelopment of brownfields and downtown revitalization, although some agricultural land still remains within the town limits (Town of Framingham, 2011).

San Marcos, representing the San Diego area and located about 35 miles north of the city, grew from a population of 17,479 in 1980 to 82,743 by 2008. Major institutions in the city include California State University San Marcos and Palomar Community College. At this stage the city is only approximately 72 percent built out, and thus new development continues (City of San Marcos, 2011).

Precipitation Data

Average monthly precipitation data were obtained from the NOAA Hourly Precipitation Data Rainfall Event Statistics[5] for one station with a long-term record in each region: Southeast—Atlanta/Hartsfield International Airport (Station #90451), South Central—Austin/Robert Mueller Municipal Airport (410428), Northeast—Boston/Logan International Airport (190770), and Southwest—San Diego/San Diego International Airport (Lindbergh Field) (47740). Atlanta receives the most precipitation, averaging about 49 inches per year, followed by Boston (47 inches/year), Austin (33 inches/year), and San Diego (10 inches/year). Figure 2 depicts precipitation variations over more than 50 years.

Values for either the 85th and 95th percentile, 24-hour storms were available in a number of state-specific resources, including the Georgia Stormwater Standards Supplement (Center for Watershed Protection 2009) and the Integrated Stormwater Management Program (North Central Texas Council of Governments 2010), as well as national publications such as an USEPA's technical guidance documents (USEPA 2009). However, few references had values for both 85th and 95th percentile storms. Therefore, these values were calculated following the methodology outlined in the USEPA's Technical Guidance on Implementing the Stormwater Runoff Requirements (USEPA 2009, page 30). Daily precipitation and temperature data from the National Climatic Data Center's TD Summary of the Day data set were collected and analyzed for the four stations lover a time period of 60 years, January 1, 1950 to January, 31 2010.

[5] National Climatic Data Center, Hourly Precipitation Data Rainfall Event Statistics (http://cdo.ncdc.noaa.gov/cgi-bin/HPD/HPDStats.pl, last accessed December 15, 2011).

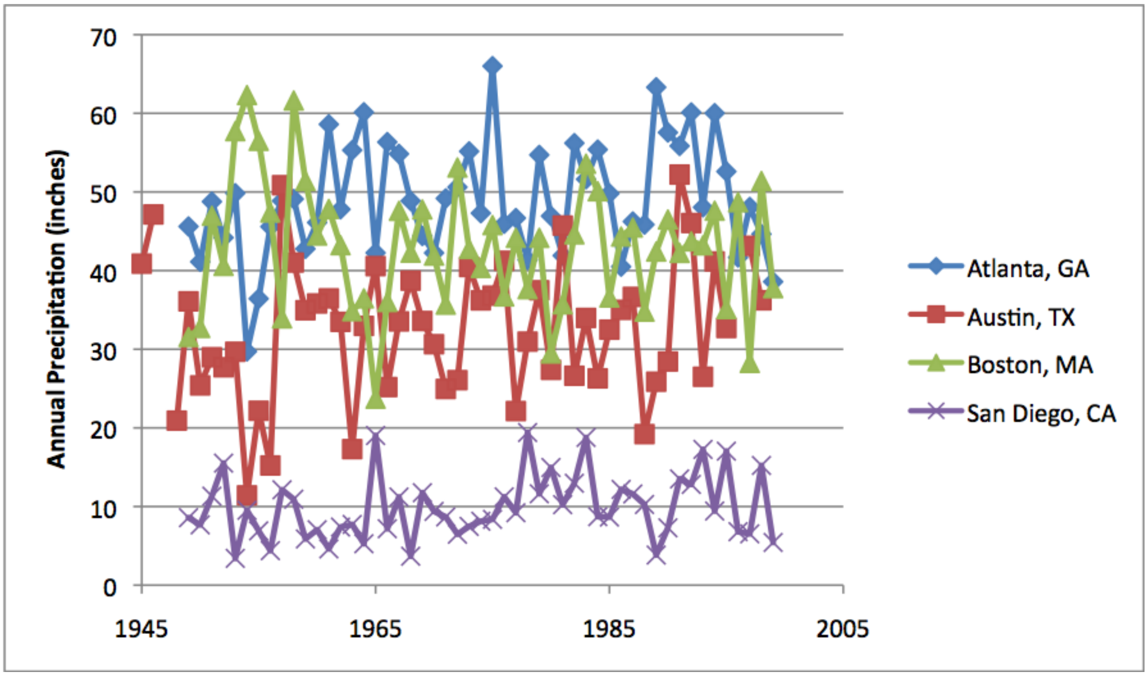

Figure 2. Average Annual Precipitation for Four Climate Regions over the Latter Part of the Twentieth Century (from NOAA Hourly Precipitation Data Rainfall Event Statistics, http://cdo.ncdc.noaa.gov/cgi-bin/HPD/HPDStats.pl)

For snowfall days, snow water equivalent (SWE) was calculated according to the guidelines provided by a National Climate Data Center's (NCDC) document, Estimating the Water Equivalent of Snow, utilizing the reported mean temperature for the day (National Climatic Data Center, accessed December 16, 2011). The NCDC tables calculate that the SWE is at most, about 10 percent of the total snowfall depth. In the methodology for determining the 85th and 95th percentile events, all days with < 0.1 inch precipitation are removed, lowering the impact of snow on the results. Snowfall had no effect in the Southwest region, a very minor effect in the Southeast and South Central, and still a relatively small effect in the Northeast, as follows: San Diego—0 snow days; Atlanta—74 of 4600 total days having ≥ 0.1 inch (1.6 percent), with a contribution ranging 0.01-0.79 inch precipitation; Austin—32 of 2418 days (1.3 percent), contributing 0.01-0.50 inch; and Boston—993 of 4783 days (20.8 percent), contributing 0.01-2.24 inch. Since snow does add to runoff that must be managed in a location like the Northeast, these snow water equivalents were left in the records. Table 3 summarizes precipitation data used in the analyses for the four regions.

Table 3. Precipitation Summary for Study Regions

Region	Average Annual Precipitation (inches)	85th Percentile, 24-Hour Event		95th Percentile, 24-Hour Event	
		Depth (inch)[a]	Fraction Covered[b]	Depth (inch)[a]	Fraction Covered[b]
Southeast	49.02	1.13	0.63	1.79	0.87
South Central	32.67	1.19	0.58	1.99	0.82
Northeast	47.03	1.07	0.81	1.72	0.89
Southwest	9.68	0.76	0.62	1.26	0.83

[a] Calculated from National Climatic Data Center's TD Summary of the Day, for all precipitation days >0.1 inch for period January 1, 1950 – December 31, 2009
[b] Fraction of total annual precipitation covered by event standard

Physiographic Data

General Methods

This section of the report covers the soils, groundwater, and topographic data underlying the analyses. Soil characteristics are largely a product of climate, geology and topography. The characteristics of most interest for this study were those controlling infiltration of surface water and percolation to an aquifer. Although there is variation within each climate region, the major soil orders can be used to identify regional characteristics. The Natural Resources Conservation Service (NRCS) website[6] describing the major soil orders and their locations was the initial source of these data. Maps generated by Miller and White (1998) gave information from the State Soil Geographic Database (STATSGO), including characteristics such as soil texture and hydrologic soil group. These resources were employed to gain a broad view of the soils in each of the four regions.

To extend the scope of the study, soils were investigated in the Upper Midwest, in addition to the Southeast, South Central, Northeast, and Southwest climate regions. Upper Midwest and Northeast soils share general similarities. Both regions also have temperate, seasonal, humid climates. While average annual precipitation is overall somewhat greater in the Northeast compared to the Upper Midwest, the two regions were deemed similar enough physiographically and climatologically to be considered together. This report henceforth groups them as the Northeast – Upper Midwest climate region.

To validate the regional patterns emerging from the general sources, custom "soil resource" reports for four cities were generated using the NRCS Web Soil Survey[7] tool. These reports collected characteristics related to infiltration rates and runoff including soil texture, hydrologic soil group, drainage classification, representative slope, and depth to water table. Using this tool requires selecting an "area of interest". This examination utilized a size of at least 8,000 acres (10,000 acres is the maximum allowed) to insure a representative sample of soil and related conditions.

Hydrologic soil group assignment is a means of generally categorizing soils according to their tendency to admit and transmit water. The hydrologic soil group (HSG) is determined with respect to the water-transmitting soil layer with the lowest saturated hydraulic conductivity and depth to any layer that is more or less water impermeable (such as a fragipan or duripan) or depth to a water table. Box 1 summarizes the characteristics of the four HSGs (NRCS 2007).

The position of the groundwater table is a crucial determinant of whether or not stormwater infiltration should be promoted by applying ground-based ARCD practices. A seasonal high water table too close to the surface results in rapid saturation of a thin soil column and retarded infiltration. Ponding water longer than 72 hours can permit mosquito growth, damage vegetation, and promote clogging of the facility by microorganism growths and polysaccharide organic materials that form in the reduced-oxygen environment accompanying excessive ponding time (Mitchell and Nevo 1964, Ronner and Wong 1996). Also, storm runoff flow through a short soil column or very rapidly through a coarse-textured soil can potentially convey contaminants to groundwater. To avoid entertaining stormwater management strategies threatening development of these problems, data on depth to groundwater was obtained from the U.S. Geological Survey's (USGS) Groundwater-Level Annual Statistics (USGS 2011).

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[6] Natural Resources Conservation Service, Distribution Maps of Dominant Soil Orders (http://soils.usda.gov/technical/classification/orders/, last accessed December 16, 2011).

[7] Natural Resources Conservation Service, 2011, Web Soil Survey (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm).

Topographic slope influences runoff production by setting incident precipitation in motion downslope, thus producing a horizontal component of velocity vector partially counteracting the tendency to penetrate the soil vertically. This study investigated that importance of that effect by considering two slopes typical of urban development sites. As discussed during the presentation of results, below, this factor did not have a large effect on the analysis.

Box 1. Summary of Hydrologic Soil Groups (NRCS 2007)

Group A—Soils in this group have low runoff potential when thoroughly wet. Water is transmitted freely through the soil. Group A soils typically have less than 10 percent clay and more than 90 percent sand or gravel and have gravel or sand textures. Some soils having loamy sand, sandy loam, loam or silt loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments. The saturated hydraulic conductivity of all soil layers exceeds 5.67 inches per hour. The depth to any water-impermeable layer is greater than 20 inches. The depth to the water table is greater than 24 inches. Soils deeper than 40 inches to a water-impermeable layer are in group A if the saturated hydraulic conductivity of all soil layers within 40 inches of the surface exceeds 1.42 inch per hour.[a]

Group B—Soils in this group have moderately low runoff potential when thoroughly wet. Water transmission through the soil is unimpeded. Group B soils typically have between 10 percent and 20 percent clay and 50 percent to 90 percent sand and have loamy sand or sandy loam textures. Some soils having loam, silt loam, silt, or sandy clay loam textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments. The saturated hydraulic conductivity in the least transmissive layer between the surface and 20 inches ranges from 10.0 1.42 to 5.67 inches per hour. The depth to any water-impermeable layer is greater than 20 inches. The depth to the water table is greater than 24 inches. Soils deeper than 40 inches to a water- impermeable layer or water table are in group B if the saturated hydraulic conductivity of all soil layers within 40 inches of the surface exceeds 0.57 inch per hour but is less than 1.42 inch per hour.

Group C—Soils in this group have moderately high runoff potential when thoroughly wet. Water transmission through the soil is somewhat restricted. Group C soils typically have between 20 percent and 40 percent clay and less than 50 percent sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures. Some soils having clay, silty clay, or sandy clay textures may be placed in this group if they are well aggregated, of low bulk density, or contain greater than 35 percent rock fragments. The saturated hydraulic conductivity in the least transmissive layer between the surface and 20 inches is between 0.14 and 1.42 inch per hour. The depth to any water-impermeable layer is greater than 20 inches. The depth to the water table is greater than 24 inches. Soils deeper than 40 inches to a restriction or water table are in group C if the saturated hydraulic conductivity of all soil layers within 40 inches of the surface exceeds 0.06 inch per hour but is less than 0.57 inch per hour.

Group D—Soils in this group have high runoff potential when thoroughly wet. Water movement through the soil is restricted or very restricted. Group D soils typically have greater than 40 percent clay, less than 50 percent sand, and have clayey textures. In some areas, they also have high shrink-swell potential. All soils with a depth to a water-impermeable layer less than 20 inches and all soils with a water table within 24 inches of the surface are in this group, although some may have a dual classification if they can be adequately drained. For soils with a water-impermeable layer at a depth between 20 and 40 inches, the saturated hydraulic conductivity in the least transmissive soil layer is less than or equal to 0.14 inch per hour. For soils deeper than 40 inches to a restriction or water table, the saturated hydraulic conductivity of all soil layers within 40 inches of the surface is less than or equal to 0.06 inch per hour.

[a] While Group A soils are present across large areas of the country, our analysis considers only Group B, C, and D soils to provide a conservative assessment of infiltration potential in urban areas, and to account for potential issues such as soil compaction that may occur for lawn and other landscaping in urban and suburban development.

Southeast Climate Region

The major soil order found throughout the southeastern United States is Utisols, sub-order Udults. The humid climate with frequent rainfall gives the soils an udic moisture regime; soils are rarely dry for more than 45 consecutive days. Utisols are highly weathered and are deficient in calcium and other bases. Georgia is known for its red soils, which are the unhydrated iron oxides left in the weathered material. Pre-European contact, these soils supported mixed conifer and deciduous woodlands. Due to its relatively flat topography and warmer temperatures, Florida has primarily Spodosols, Alphisols and Histosols (Soil Survey Staff, NRCS 2011).

This region has a variety of soil textures, ranging from sand and sandy loam throughout Mississippi, Alabama, and Georgia; silty loam soils near the Appalachian Mountains; and some areas with significant organic materials in Florida. The major soil hydrologic groups of the region are varied as well, with C and D soils dominating the Georgia coastline and most of Florida. Group A and B soils are more prevalent in the interior parts of the region, in central Georgia and Alabama (Miller and White 1998).

A NRCS web soil survey was conducted for an area of interest (AOI) centered in Alpharetta, GA. The selected AOI did not have complete soil survey coverage, and findings were compared with another AOI of 8990.5 acres north of the city in Fulton County. In both AOIs, the leading HSG is B (86 percent of AOI), followed by group C (11 percent of AOI). Approximately 97 percent of the AOI has a sandy loam soil texture. The leading drainage classification was well drained (86 percent of AOI), followed by somewhat poorly drained (10 percent of AOI). The selected AOI was moderately steep, with approximately 70 percent of the AOI having slopes between 8 and 12 percent.

Fulton County, Georgia has four wells in the USGS record, three with depth-to-groundwater data. Two wells have only one recorded depth: site 08CC08 had a depth of 2.447 ft in 1986, and site 10DD01 had a depth of 16.131 ft in 1968. Site 10DD02 has been monitored annually from 1977-2010 and has an annual well-depth average in this time period of 6.292 ft.

South Central Climate Region

The major soil order in Texas is Mollisols, sub-order ustolls. These soils span the sub-humid and semiarid climate zones, and are common on the western Great Plains and throughout the Rocky Mountain States. These soils originally supported grasslands and (in mountainous regions) forests, and now are ranched or farmed. Houston black soils are also characteristic of the region and are important in agriculture and urban areas, occurring throughout central Texas. Dry soils in the Order Aridisols, sub-orders Argids and Calcids, are found in west Texas and large portions of New Mexico as well. These soils were formerly sparsely vegetated areas, now used for rangeland or wildlife habitat (Soil Survey Staff, NRCS 2011).

Soil characteristic maps generated by Miller & White (1998) indicate that the majority of soil types in the South Central climate region are diverse: sandy loam and clay dominate eastern Texas, clay soils are prevalent in central parts of the state and loam soils are in western Texas and New Mexico. Most soils tend to be in the C and D hydrologic groups, however B soils are found in bands in New Mexico (Miller & White, 1998).

A web soil survey was conducted for an area of interest of 8267.5 acres centered in Round Rock, TX. The leading HSG is D (68 percent of AOI), followed by group C (22 percent of AOI) and group B (10 percent). Primary soil textures are clay (33 percent), silty clay (27 percent), extremely stony clay (17 percent), and silty clay loam (10 percent). The leading drainage classification is well drained (79 percent of AOI) followed by moderately well drained (21 percent). The selected AOI is relatively flat; approximately 70 percent of the AOI has slopes under 2 percent, and 20 percent has slopes of 3-4 percent.

Travis County, Texas had three wells that were measured in 2003 and recorded by USGS (site YD-58-50-216) and 2004 (sites YD-58-50-216 and YD-58-25-907). Groundwater is very deep in each location, averaging 220 ft below the ground surface.

Northeast – Upper Midwest Climate Region

This climate region has significant variation in dominant soil orders. The Spodsols order, sub-order Orthods, dominates the northern portions (northern Minnesota, Wisconsin, Michigan, Vermont, and Maine) and is generally considered infertile without soil amendments. Inceptisols, sub-order Udepts, are also prevalent in the region, especially in New England states, through the Appalachian Mountains and northeastern Minnesota. Alfisols, sub-order Udalfs, too are prevalent in the region, extending from Minnesota east to New York. These two soils both have an udic moisture regime, and are rarely dry for more than 45 consecutive days due to the year-round precipitation in the area (Soil Survey Staff, NRCS 2011). The state soil of Massachusetts is the Paxton fine sandy loam and also extends into New Hampshire, New York and Vermont. These deep soils were formed in acid subglacial till and are derived from schist, gneiss and granite (NRCS undated).

Based on maps generated by Miller and White (1998), sandy loam and silt loam soils tend to dominate the region, with small areas of clay and silty clay soils. Hydrologic soil group B is most prevalent in the Midwestern states (Minnesota, Wisconsin, Illinois), and Group C is most common in the rest of the region, spanning from Indiana to Maine. The region primarily supported forest ecosystems before development.

A web soil survey was conducted for an area of interest centered in Framingham, MA with an AOI of 8645.6 acres. The region has relatively equal amounts of each HSG: 20 percent of the AOI in Group A, 19 percent in group B, 20 percent in Group C, and 24 percent in Group D. Soil textures represented are fine sandy loam (49 percent), muck (10 percent), loamy sand (9 percent), and moderately decomposed plant material (8 percent). The leading drainage classification is well drained (32 percent of AOI) followed by very poorly drained (16 percent), somewhat excessively drained (12 percent), and moderately well drained (11 percent). Fourteen percent of the AOI has slopes of 1 percent or less, with 18 percent at 2-5 percent, 23 percent at 6-8 percent, and another 23 percent at 8-12 percent slopes.

There are three wells in the USGS record for Middlesex County, MA including 5 years of record for an Acton well averaging 17.75 ft, 6 years for the Wakefield well with an average depth of 6.59 ft, and 11 years at the Wilmington well with an average of 8.09 ft.

Southwest Climate Region

There are multiple soil orders in California due to its variation in climate, topography and geologic history. Entisols occur in the southern parts of the state; sub-order Psamments is a frequently found sandy soil that makes productive rangeland. Order Mollisols, sub-order Xerolls, are freely drained and dry soils found in the Mediterranean climate along the coast of California. Pre-settlement ecosystems supported by these soils include oak savanna, grasslands, and chaparral. Current soils may be used as cropland or rangeland (Soil Survey Staff, NRCS 2011).

A web soil survey was conducted for an 8267.5-acre area of interest centered in San Marcos, CA. The leading HSG is D (58 percent of AOI), followed by group C (26 percent) and group B (14 percent). Soil texture include sandy loam (19 percent), coarse sandy loam (17 percent), silt loam (15 percent), very fine sandy loam (14 percent), loamy fine sand (12 percent), loam (7 percent), and clay (5 percent). The leading drainage classification is well drained (51 percent of AOI), followed by moderately well drained (34 percent). Approximately 10 percent of the AOI has slopes ≤ 5 percent, and 66 percent has slopes of 5-10 percent.

There are no groundwater records for San Diego County available on the USGS website. Data were collected from the California Department of Water Resource Water Data Library[8]. Ten wells west of San Marcos near Escondido were sampled in 1987. The depth to groundwater ranged from 2.0 to 28.1 ft for an average of 11.6 ft.

Summary of Physiographic Characteristics

Due to the large area of land encompassed in each climate region, it is difficult to select one location that is truly "representative" of the entire region. By selecting four cities that are spaced throughout the country with different climate and soil characteristics, however, this study can demonstrate the different potential for ARCD strategies in regions around the nation. Table 4 summarizes the major soils, groundwater, and topographic characteristics for these regions. Figure 3 shows the distributions of hydrologic soil groups in areas of interest investigated in the four metropolitan areas.

Table 4. Summary of Physiographic Data

Characteristic	Southeast	South Central	Northeast – Upper Midwest	Southwest
Main soil types	Sandy loam	Clay, clay loam	Sandy loam, silt loam	Sandy loam, loam
Hydrologic soil group near study site	B (GA, AL, SC)	D (TX)	C (Northeastern states)	D
Other hydrologic soil group in climate region	D (FL)	C (NM)	B (MN, WI, IL, MI)	C
Predominant pre-development land cover	Woods	Semi-arid herbaceous	Woods	Narrow-leaved chaparral
Predominant slopes	70% @ 8-12%	90% < 4%	65% < 12%	76% < 10%

LAND USE CASES

Five cases were selected to represent a range of urban development types considered to be representative of the nation. These cases involved: a multi-family residential complex (MFR), a relatively small-scale (23 homes) single-family residential development (Sm-SFR), a relatively large (1000 homes) single-family residential development (Lg-SFR), a sizeable commercial retail installation (COMM), and an urban redevelopment (REDEV).

Building permit records from the City of San Marcos in San Diego County, California provided data on total site areas for the first three cases, including numbers of buildings, building footprint areas (including porch and garage for Sm-SFR), and numbers of parking spaces associated with the development projects. Information was not as complete for cities in other regions, but what data was available indicated no substantial difference in these site features. Therefore, the San Marcos data were used for all regional case studies. This uniformity had the advantage of placing comparisons completely on the basis of the major variables of interest, climatological and soils characteristics.

[8] http://www.water.ca.gov/waterdatalibrary (last accessed December 16, 2011).

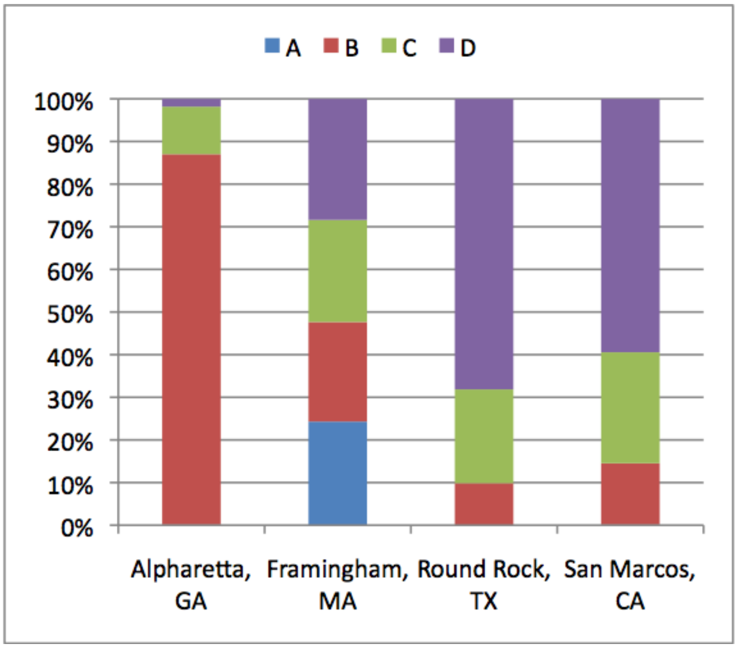


Figure 3. Distribution of Hydrologic Soil Groups in Four Study Cities

The REDEV case was taken from an actual project in Berkeley, California involving conversion of an existing structure, built originally as a corner grocery store, to apartments and addition of a new building to create a nine-unit, mixed-use, urban infill project. Space remained for a large side yard.

Larger developments were not represented in the sampling of building permits from the San Marcos database. To take larger development projects into account in the subsequent analysis, the two larger scale cases were hypothesized. The Lg-SFR scenario scaled up all land use estimates from the Sm-SFR case in the ratio of 1000:23. The hypothetical COMM scenario consisted of a building with a 2-acre footprint and 500 parking spaces. As with the smaller-scale cases, these hypothetical developments were assumed to have roadways, walkways, and landscaping, as described below.

While the building permit records made no reference to features such as roadways, walkways, and landscaping normally associated with development projects, these features were taken into account in the case studies using assumptions described herein. Parking spaces were estimated to be 176 square ft in area, which corresponds to 8 ft width by 22 ft length dimensions. Code requirements vary by jurisdiction, with the tendency now to drop below the traditional 200 square ft average. About 180 square ft is common, but various standards for full- and compact-car spaces, and for the mix of the two, can raise or lower the average (Gibbons, 2009). The 176 square ft size is considered to be a reasonable value for conventional practice.

Roadways and walkways assume a wide variety of patterns. Exclusive of the two SFR cases, simple, square parking lots with roadways around the four sides and square buildings with walkways also around the four sides were assumed. Roadways and walkways were taken to be 20 ft and 6 ft wide, respectively.

Each single-family residences (SFR) was assumed to have a lot area of 5749 square ft,, and a driveway 20 ft wide and 30 ft long. Assuming a square lot, each would have a sidewalk 76 feet by 4 feet wide, and a walkway that is 40 feet by 4 feet. .

Exclusive of the COMM case, the total area for all of these impervious features was subtracted from the total site area to estimate the pervious area, which was assumed to have conventional landscaping cover (grass, small herbaceous decorative plants, bushes, and a few trees). For the COMM scenario, an additional 10 percent was added to the building, parking lot, access road, and walkway area to represent the landscaping, on the belief that a typical retail commercial establishment would be mostly impervious.

Table 5 summarizes the characteristics of the five land use cases. The table also provides the recorded or estimated areas in each land use and cover type.

Table 5. Summary of Cases with Land Use and Land Cover Areas

	MFR[a]	Sm-SFR[a]	Lg-SFR[a]	COMM[a]	REDEV[a]
No. buildings	11	23	1000	1	2
Total area (ft^2)	476,982	132,227	5,749,000	226,529	5,451
Roof area (ft^2)	184,338	34,949	1,519,522	87,120	3,435
No. parking spaces[b]	438	-	-	500	2
Parking area (ft^2) [b]	77,088	-	-	88,000	316
Access road area (ft^2)	22,212	-	-	23,732	-
Walkway area (ft^2)	33,960	10,656	463,289	7,084	350
Driveway area (ft^2)	-	13,800	600,000	-	650
Landscape area (ft^2)	159,384	72,822	3,166,190	20,594	700

[a] MFR—multi-family residential; Sm-SFR—small-scale single-family residential; Lg-SFR—large-scale single-family residential; COMM—retail commercial; REDEV—redevelopment
[b] Uncovered

METHODS OF ANALYSIS

AVERAGE EVENT AND ANNUAL STORMWATER RUNOFF VOLUMES

Calculation Methods

Surface runoff volumes produced were estimated for both pre- and post-development conditions for each case study. The pre-development state was considered to be the predominant land cover for each region prior to European settlement.

For impervious areas, average event and annual runoff volumes were computed as the product of event or average annual precipitation, contributing drainage area, and a runoff coefficient (ratio of runoff produced to precipitation received) according to the familiar Rational Method equation. The runoff coefficient was determined from the equation C = (0.009) I + 0.05, where I is the impervious percentage. This equation was derived by Schueler (1987) from Nationwide Urban Runoff Program data (USEPA 1983). With I = 100 percent for fully impervious surfaces, C is 0.95.

The basis for pervious area runoff coefficients, for both the pre-development state and landscaped areas in developments, was the NRCS's Urban Hydrology for Small Watersheds (NRCS 1986, as revised from the original 1975 edition). This model estimates storm event runoff (R, inch) as a function of precipitation (P, inch) and a variable representing land cover and soil, termed the curve number (CN, dimensionless). CN enters the calculation via a variable S, which is the potential maximum soil moisture retention after runoff begins. The equations for English units of measurement are:

$$R = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad\qquad S = \frac{1000}{CN} - 10$$

The runoff equation is valid for P > 0.2S, which represents the initial abstraction, the amount of water retained before runoff begins by vegetative interception and infiltration (NRCS 1986). According to this model, larger events are forecast to produce a greater amount of runoff in relation to amount of precipitation, because they more fully saturate the soil. Therefore, use of the model to estimate annual runoff requires selecting some event or group of events to compute an average runoff coefficient representing the year.

Average pre- and post-development pervious area average runoff coefficients were derived by computing runoff from a series of precipitation events ranging from 0.1 inch up to the 95th percentile, 24-hour event for the respective metropolitan areas, dividing by the associated precipitation, and averaging for all event amounts > 0.2S. Average annual runoff volumes for pervious areas were estimated based on these runoff coefficients and average annual precipitation quantities recorded at the respective gauging locations.

Curve Number Selection

Pre-development curve numbers were determined from existing studies and NRCS (1986) CN tables based on pre-European settlement land cover. Before development, woods predominated in Georgia and Massachusetts. Pre-development Texas had principally arid and semi-arid range with herbaceous cover. Chaparral was the predominant land cover in the San Diego area, however, this land cover type is not listed in the NRCS tables. For that region the selection came from a study by Easterbrook (undated) on curve numbers and associated soil hydrologic groups in an investigation of mainly chaparral lands before and after wildfires in the San Diego area.

Conversion to landscaping typical of development modifies soil and water infiltration characteristics by removing topsoil and even subsoil, compacting the remaining soil, and changing the vegetative cover. For pervious landscaping after development, CN was based on 1/8-acre urban development for all building types.

To demonstrate a range of results, runoff estimates were made for two soils in each region falling in B and C, B and D, or C and D HSGs. The more infiltrative soil was assumed to be in "good" condition and the less permeable one in "poor" condition, differentiations made in the NRCS tables. Table 6 summarizes the curve numbers used in the analyses. The paragraphs following the table detail how the selections were made for each region.

Table 6. Summary of Curve Numbers for Study Regions

	Southeast		South Central		Northeast – Upper Midwest		Southwest	
Hydrologic soil group-condition	B-good	D-poor	C-good	D-poor	B-good	C-poor	C-good	D-poor
Pre-development	55	83	74	93	55	77	77	90
Post-development	85	92	90	93	85	90	91	93

The Georgia Stormwater Manual Supplement recommends that watershed managers select curve numbers proposed by the NRCS based on hydrologic soil groups A through D and hydrologic condition of the site (Center for Watershed Protection 2009). As aforementioned, the pre-European land cover of the southeastern United States was forested. A study by Dyke (2001) in Forsyth and Hall Counties northeast of Atlanta confirmed that, immediately prior to development, approximately 50 percent of urban lands were forested, with 22 percent in agricultural use.

Because the region includes B soils in the interior of Alabama and Georgia, and poorly draining D soils in Florida and along the coasts, it was decided, for the purpose of demonstrating a range of results, to base NRCS Curve number values on B soils in good condition and D soils in poor condition. The corresponding pre- and post-development curve numbers are 55 and 83 and 85 and 92, respectively.

Prior to human development, approximately 80 percent of Texas, mostly in the central part, was covered in short and tall grassland communities; the western 10 percent of the state was desert grassland; and the eastern 10 percent was forested (University of Texas 2000). McLendon (2002) conducted a study on the observed and predicted curve numbers in 107 watersheds in Texas. For rural watersheds the CNs ranged from 48 to 88. The range in Austin was 49-89 and in Dallas 60-90. The Texas Department of Transportation's (2001) Hydraulic Design Manual Section 7 lists values for pre-development curve numbers for arid and semi- arid rangelands. Based on these sources, the respective pre- and post-development CN choices were 74 (C—good soil) and 93 (D—poor soil) and 90 (C—good soil) and 93 (D—poor soil).

Before European development, most of the Northeast – Upper Midwest region was covered in mixed hardwood and coniferous forests. A recent USGS report confirms that most urban development in the region from 1973 to 2000 has converted forestland (47 percent of all changes), followed by farmland (11 percent) (Auch undated). For this study's pre-development curve number, the woods cover type, soil group B in good condition and C soil in poor condition gave corresponding curve numbers of 55 and 77, respectively. Post-development curve numbers for these soil types at 1/8-acre development size were 85 and 90 for the good B and poor C soils, respectively. These post-development curve numbers are similar to a recent study in the Aberjona River watershed, an urban catchment northwest of Boston, where the authors used an overall CN of 89 to represent the more impervious parts of the watershed (Perez-Pedini et al. 2005).

With the lack of NRCS data for chaparral, CN selection for the San Diego area was based on an analysis performed in the area of the 2003 Cedar Fire in San Diego County by Easterbrook (undated). For pre-development C soils in good condition and D soils in poor condition, the choices were 77 and 90, respectively. Post-development curve numbers were selected from Easterbrook's estimation of CN after a high-burn fire; for good C soils CN = 91, and for poor D soils CN = 93.

Effect of Slope on Curve Number

NRCS documents developing the curve number concept and associated methods did not cover the effect of land slope. Independent researchers have given some attention to the question though. Sharpley and Williams (1990) introduced the empirical equation that has been most often used to adjust CN relative to slope:

$$CN_s = 0.333(CN_w - CN)(1 - 2e^{-13.86s}) + CN$$

where CN is the curve number reported in NRCS tables for an average soil moisture condition and assumed slope ≤ 5 percent, CN_s = slope-adjusted CN, CN_w = CN in an initially wet soil condition, and s = slope (ft/ft). Ward and Trimble provided factors to adjust tabulated CN values to obtain CN_w. Carrying through the analysis in this manner demonstrated that results deviated between two assessed slopes (5 and 10 percent) by only around 2-6 percent. This small difference was considered minimal in the context of the approximations and assumptions inherent in the modeling process. While the results presentation gives some additional data on slope effects, full coverage is given only for 5 percent, the topographic basis of the NRCS model and by far the subject of its greatest application.

ESTIMATING INFILTRATION CAPACITY OF THE CASE STUDY SITES

Infiltration Rates

Infiltrating sufficient runoff to maintain pre-development hydrologic characteristics and prevent pollutant transport is the most effective way to protect surface receiving waters. Successfully applying infiltration requires soils and hydrogeological conditions that will pass water sufficiently rapidly to avoid overly-lengthy ponding, while not allowing percolating water to reach groundwater before the soil column captures pollutants.

The study assumed that infiltration would occur in surface facilities and not in below-ground trenches. The use of trenches is certainly possible. However, the intent of this investigation was to determine the ability of pervious areas to manage the site runoff, and their exclusion is consistent with the conservative approach to modeling taken in this analysis. This inquiry was accomplished by evaluating the ability of the predominant soil types identified for each region to provide an infiltration rate of at least 0.5 inch/hour, the rate often regarded in the stormwater management field as the minimum for the use of infiltration practices (e.g., Geosyntec Consultants 2008). The assessment considered soils that either would provide this rate, at a minimum, in their original condition or could be organically amended to augment soil water storage and increase infiltration, while also safeguarding groundwater. Therefore, prevailing groundwater depths were assessed in relation to runoff percolation times generally regarded as safe.

Infiltration rates were based on saturated hydraulic conductivities (obtained from Leij et al. 1996) typical of the basic soil types incorporated in the U.S. Department of Agriculture (USDA, 1987) soil textural triangle. Sand, loamy sand, sandy loam have conductivities well above 0.5 inch/hour. As Table 4 indicates, three of the four regions have a sandy loam as the dominant soil type. For such a soil in the B HSG in these regions, the infiltration rate was taken as 1.74

inch/hour (Leij et al. 1996). Other textures represented that would generally fall in the C group are mostly loam and silt loam. These soil types either have conductivities in excess of 0.5 inch/hour or, in the first author's experience, can be and have been successfully organically amended to produce such a rate and infiltrate accumulated water within 72 hours, and usually less time. The D soils in some study regions, silty clay and clay, were regarded as not amendable to reach 0.5 inch/hour conductivity to host conventional or ARCD-type facilities designed specifically for infiltration. Still, locations with these soils could distribute sheet flow over pervious areas for evapotranspiration and some infiltration at slow rates and could utilize roof downspout surface or subsurface dispersal.

Groundwater Protection Assessment

Avoidance of groundwater contamination was assessed by assuming a hydraulic conductivity generally regarded as the maximum rate for the use of infiltration practices, 2.4 inches/hour (e.g., Geosyntec Consultants 2008), and a minimum spacing to seasonal high groundwater from the bed of an infiltration facility of 4 ft. These conditions would provide a travel time of 20 hours, during which contaminant capture would occur through soil contact. This 20-hour travel time was regarded as a minimum for any soil type. For example, infiltrating on loamy sand with a hydraulic conductivity of 5.7 inches/hour would require minimum spacing from the infiltration surface to groundwater of 10 ft. This consideration did not actually become an issue for analyses in any region in this study, because all predominant soil types have infiltration rates under 2.4 inches/hour and groundwater spacings that exceed 4 ft.

Site Infiltration Capacities

Runoff volumes were estimated for the 85th and 95th percentile, 24-hour events as described previously. Bioretention cell surface area to accommodate these volumes was calculated based on a method in the City of Santa Barbara's Storm Water BMP Guidance Manual (Geosyntec Consultants 2008) (adapted from the Georgia Stormwater Manual (Atlanta Regional Commission, 2001)):

$$A = \frac{(V_{design})(l)}{(t)(k_{design})(d+l)}$$

where:

V_{design} = design volume of runoff to be infiltrated (ft3);

k_{design} = design infiltration rate (in/hr), taken as 0.5 times the typical rate for the soil type naturally or amended as a safety factor;

d = ponding depth (ft), assumed as 0.25 ft for a shallow landscape feature on the recommendation of the Georgia manual;

l = depth of planting media (ft), assumed as 4 ft on the recommendation of the Georgia manual;

t = required drawdown time (hr), taken as 48 hours.

The design variable selections are conservative in applying a safety factor to hydraulic conductivity, using minimum depths for economy and limiting site disruption, and applying a drain time lower than the maximum of 72 hours.

In considering the long-term capacity of a facility designed to infiltrate, the potential for groundwater mounding below or aside the unit is a concern. To avoid this problem a basic analysis was made using a groundwater rise equation from Zomorodi (2005):

$$\text{Rise} = 0.86\,\frac{(K_v)(W)}{(K_h - K_v)}$$

where:

Rise = mounding occurring in a year of use (ft);

K_v = vertical saturated hydraulic conductivity (ft/year);

W = bioretention cell width (ft); and

K_h = horizontal saturated hydraulic conductivity (ft/year).

This equation was solved for K_v for computation of the allowable annual infiltration rate, assuming a rise limited to 1 ft. It was assumed that the bioretention surface area would be broken up to have no more than one basin for each 5 acres of total site area, another measure safeguarding against groundwater mounding. Also assumed was a square cell (i.e., W was computed as the square root of the surface area calculated according to the equation for A above). Horizontal hydraulic conductivites for loams such as represented among the B and C soils in the study regions tend to run in the range of 10 to 1000 meters/year (0.1 to 9 ft/day. A conservative value of 3 ft/day was used in the analysis.

The yearly rate of infiltration from a bioretention cell can be expressed in terms of volume of runoff per unit infiltrating surface area, acre-ft/acre-year, which is equivalent to K_v expressed as ft/year. The K_v value avoiding groundwater monitoring was therefore used to assess maximum annual infiltration capacity by multiplying by the total available pervious surface area. However, the K_v value was capped at a rate found in a study of infiltration capacity and benefits for Los Angeles' San Fernando Valley by Chralowicz et al. (2001). The Los Angeles study posited providing 0.1-0.5 acre for infiltration basins to serve each 5 acres of contributing drainage area. At 2-3 ft deep, it was estimated that such basins could infiltrate 0.90-1.87 acre-ft/year of runoff in San Fernando Valley conditions. Three types of soils predominate in the study area: sandy loams (35 percent of the area), a clay loam (23 percent), and a silty clay loam (29 percent). The balance of 13 percent includes small amounts at both ends of the textural spectrum, a clay and loamy sands. Infiltration rates are in the approximate range of 0.5-2.0 inches/hour, within the span generally regarded as ideal for successful infiltration without threatening groundwater. Computing the ratios of the rate and basin size data of Chralowicz et al. (2001), K_v maximized at approximately 20 acre-ft of runoff/acre infiltration surface-year under the most limiting conditions of soils and basin dimensions. This value was applied in this study if calculated rates were higher, another conservative feature to obtain the most realistic projections of infiltration potential.

In some cases analyzed, the maximum annual infiltration capacity was estimated at greater than post-development runoff volume production. In these instances complete retention would be possible with excess capacity left, and only a fraction of the available pervious area would have to be devoted to bioretention. That fraction was expressed as the ratio of annual runoff production to infiltration capacity.

STORMWATER RUNOFF VOLUME AND POLLUTANT DISCHARGES

Urban Land Use Pollutant Yields

Annual pollutant mass loadings prior to application of any stormwater management practices were estimated as the product of annual runoff volumes produced by the various land use and cover types and pollutant concentrations typical of those areas. General land use types (e.g., single-family residential, commercial) have typically been the basis for measuring and reporting stormwater pollutant data. However, an investigation of ARCD practices of the type of interest in this study demands data on specific land coverages. The literature offers few data on this basis. Those available and used herein were assembled by a consultant to the City of Seattle for a project in which the author participated. They appear in Attachment A (Herrera Environmental Consultants, Inc. undated). Table 7 summarizes the representative values used in the analysis.

Table 7. Pollutant Concentrations in Runoff from Developed Land Uses (after Herrera Environmental Consultants, Inc. undated)

Land Use	Total Suspended Solids (mg/L)	Total Copper (µg/L)	Total Zinc (µg/L)	Total Phosphorus (µg/L)
Residential roof	25	13	159	110
Commercial roof	18	14	281	140
Access road/driveway	120	22	118	660
Parking	75	36	97	140
Walkway	25	13	59	110
Landscaping	213	13	59	2040

Pollutant concentrations expected to occur typically in the mixed runoff from the several land use and cover types making up a development were estimated by mass balance; i.e., the concentrations from the different areas of the sites were combined in proportion to their contribution to the total runoff.

Estimating Retention

The principal interest of this study was to estimate how much of the post-development runoff volume for the various land use cases could be retained by ARCD measures and prevented from discharging from the site on the surface. The analyses initially evaluated the runoff volume that could potentially be infiltrated by using a portion or all of the available pervious area for bioretention facilities. In some instances judicious use of the pervious area could infiltrate the full volume. In other cases use of the pervious area for as much infiltration as possible plus special management of roof runoff would fully attenuate post-development runoff.

Complete retention would, of course, exceed any ordinary regulatory standard intended to govern discharge quantity and quality. To the extent that full retention could not be expected, the study was interested in assessing the degree to which bioretention and roof runoff management could meet the specific potential standards outlined earlier. Performance was estimated in terms of volume retained versus released, the extent to which pre-development groundwater recharge would be preserved, and the pollutant loading reduction accompanying volume retention in comparison to the quantities that would enter receiving waters with no stormwater management actions. These measures expressed in equation form are:

$$\text{Runoff retention (\%)} = \frac{(\text{Volume with no practices} - \text{Volume with ARCD practices})}{\text{Volume with no practices}} x100$$

(expresses amount of the theoretical maximum post-development runoff prevented from discharging by ARCD)

$$\text{Recharge retention (\%)} = [1 - \frac{(\text{Predevelopment recharge} - \text{Postdevelopment recharge with ARCD})}{\text{Predevelopment recharge}}] x100$$

Pre-development recharge = Rainfall volume – Predevelopment runoff volume

Post-development recharge = The smaller of rainfall volume or post-development infiltration volume

$$\text{Loading reduction (\%)} = \frac{(\text{Loading with no practices} - \text{Loading with ARCD practices})}{\text{Loading with no practices}} x100$$

It should be noted that runoff retention and recharge retention express different quantities and are not equal numerically.

When infiltration alone (Basic ARCD) could not accomplish full retention, roof runoff management strategies were selected as appropriate for the land use case (Full ARCD). For the retail commercial development (COMM), roof runoff management was assumed to be accomplished by harvesting, temporarily storing, and applying water to use in the building. To this end, the assumption was made that the commercial development would be able to manage and would have capacity to store and make use of the entire roof runoff volume. While this particular assumption is, on its own, speculative, the commercial development would, as discussed in the section on Application of ARCD Practices, earlier, see a reduction in runoff as a result of evapotranspiration, and would have the option to employ ARCD site design principles to reduce impervious surface area, to install a green roof to retain runoff, or to implement any of a number of other ARCD practices designed to reduce runoff volume and pollutant loading. As a result, the overall analysis of the commercial site remains conservative in its assessment of the potential to retain runoff onsite.

In the three multi-family and single-family residential cases it was assumed that the roof water would be dispersed on or within the pervious area according to accepted and standardized practices. For example, the Washington Department of Ecology's (2005) Stormwater Management Manual for Western Washington provides design criteria for two methods: splash blocks followed by vegetated dispersion areas and gravel-filled trenches. These devices can be used wherever space is sufficient regardless of infiltration rates, as they operate by evapotranspiration and slow infiltration. Even clay can infiltrate at an approximate rate of 0.2 inch/hour or higher (Leij et al. 1996; Pitt, Chen, and Clark 2002). Care was taken to assure that pervious area already allocated to infiltration would not also be counted upon for dispersion. While dispersion was assumed for simplification of the study analyses, in reality a site designer would have the option of using rain barrels, cisterns, and/or green roofs instead of or along with ground dispersion to manage roof water. Analyses for the final case, the redevelopment scenario (REDEV), assumed dispersion and/or small-scale harvesting of roof runoff above whatever level of infiltration could be accomplished given the soil condition.

Additional Analyses When Full Retention Cannot Be Expected

Retaining runoff from impervious and pollutant generating pervious surfaces is the best stormwater management policy, because it prevents the introduction of urban runoff pollutants to receiving waters as well as serves quantity discharge control requirements. Maintaining pre-development peak flow rates, volumes, and elevated flow durations prevents stream channel and habitat damage, flooding, and loss of groundwater recharge. When conditions were expected to render full retention technically infeasible for the study cases, estimates were made of the volume and pollutant loadings that would be discharged assuming the remaining surface runoff is released to a receiving water with and without treatment. Treatment was assumed to be provided by bioretention discharging either directly on the surface or via an underdrain. While not as environmentally beneficial as retention, such treatment is superior to conventional stormwater management practices like ponds and sand filters. It captures pollutants through a number of mechanisms as contaminants are held for a time in the facility and contact vegetation and soil, such as sedimentation, filtration by plants, and adsorption and ion exchange in soil.

The effectiveness of bioretention in removing pollutants from surface runoff was estimated according to measurements by Chapman and Horner (2010). This study was performed on a linear bioretention device located on a slope and made up of a number of cells separated by weirs (termed a “cascade”). While an estimated 74 percent of all entering runoff infiltrated or evapotranspired before discharging, the flows reaching the end in the larger storms would have less residence time in the facility than in a unit on flat ground percolating water through soil before surface discharge via an underdrain. Therefore, pollutant concentrations exiting such a unit could be less yet. On the other hand, some bioretention facilities bypass the relatively rare higher flows, affording no treatment, while the cascade was designed to convey all runoff, even beyond its water quality design storm flow, and provide some treatment. On balance between the advantage and disadvantage of the facility providing the data, the discharge concentrations are considered to be representative of bioretention.

Chapman and Horner (2010) computed volume-weighted average discharge pollutant concentrations by multiplying concentrations times flow volumes for each monitored storm, summing, and dividing by total volume. The resulting values for the contaminants considered in this study are: total suspended solids (TSS)—30 mg/L, total copper—6.3 µg/L, total zinc—47 µg/L, and total phosphorus—133 µg/L. In a few instances these concentrations are higher than those in Table 7, an expression of the observation sometimes made in stormwater management that treatment cannot reduce concentrations in relatively “clean” flows below certain minimum values. In these situations the concentrations in Table 8 were also used in computing discharge loadings; i.e., no concentration reduction was applied in estimating discharge loadings, although flow volume would still be decreased to the extent infiltration could occur.

RESULTS OF THE ANALYSIS

ASSESSMENT OF MAXIMUM ARCD CAPABILITIES

Runoff Retention and Groundwater Recharge

Basic ARCD

One goal of this exercise was to determine if ARCD practices could eliminate post-development runoff production, and the pollutants it transports, and maintain pre-development groundwater recharge. The first assessment, termed the Basic ARCD analysis in this report, was to estimate if each site's pervious area is sufficient for full infiltration if given to this purpose to the extent necessary without compromising other uses. Accordingly, shallow, unobtrusive bioretention cells (i.e., rain gardens) are envisioned, dispersed through sites at no more than one for each 5 acres. It bears reemphasis that no credit was taken for water loss through evapotranspiration in this assessment, although a substantial, but not necessarily easily quantifiable, amount would undoubtedly occur. Estimates of runoff retention are therefore conservative.

Table 8 presents comparisons, for the Southeast climate region, between estimated annual runoff volumes generated before development and then post-development with and without Basic ARCD stormwater management. The table also gives annual groundwater recharge estimates for these same conditions.

Table 8. Runoff and Groundwater Recharge Volumes with Basic ARCD: Southeast Climate Region[a]

Period	Volume (acre-ft) or Percentage Measure	MFR	Sm-SFR	Lg-SFR	COMM	REDEV
B soil						
Pre-dev.	Runoff	0.046	0.013	0.56	0.022	0.001
	Recharge	44.7	12.4	539	21.2	0.51
Post-dev.	Runoff without stormwater practices	29.5	6.85	298	18.7	0.45
	Runoff retained with Basic ARCD	29.5	6.85	298	8.30	0.21
	Runoff released with Basic ARCD	0	0	0	10.4	0.25
	Runoff retention (%)	100%	100%	100%	44%	45%
	Recharge without stormwater practices	15.3	5.55	241	2.53	0.06
	Recharge with Basic ARCD	44.7	12.4	539	8.30	0.21
	Recharge retention (%)	100%	100	100%	39%	40%
	Pervious area needed (%)[b]	36%	22%	22%	100%	100%
D soil						
Pre-dev.	Runoff	13.5	3.76	163	6.43	0.16
	Recharge	31.2	8.64	376	14.8	0.36
Post-dev.	Runoff without stormwater practices	Full ARCD needed to maximize retention on D soil				
	Runoff retained with Basic ARCD					
	Runoff released with Basic ARCD					
	Runoff retention (%)					
	Recharge without stormwater practices	11.6	4.17	181	2.12	0.05
	Recharge with Basic ARCD	Full ARCD needed to maximize retention on D soil				
	Recharge retention (%)	37%	48%	48%	14%	14%
	Pervious area needed (%)[b]	Full ARCD needed to maximize retention on D soil				

[a] Pre-dev.—pre-development; post-dev.—post-development; ARCD—aquatic resources conservation design; MFR—multi-family residential; Sm-SFR—small-scale single-family residential; Lg-SFR—large-scale single-family residential; COMM—retail commercial; REDEV—infill redevelopment; Basic ARCD—infiltrating bioretention; runoff—quantity of water discharged from the site on the surface; recharge--quantity of water infiltrating the soil

[b] Proportion of the total pervious area on the site required for bioretention to achieve given results

In all cases the majority of the infiltration that would recharge groundwater in the undeveloped state would be lost to surface runoff after development. These losses would approach 90 percent in the most impervious developments. The greatly increased surface flow would raise peak flow rates and volumes in receiving water courses, increase flooding risk, and transport pollutants.

Basic ARCD could retain all post-development runoff and pre-existing groundwater recharge in the three residential cases on the B soils, using from less than one-fourth to just over one-third of the available pervious area for bioretention cells. Taking all available pervious area for the more highly impervious COMM and REDEV cases on B soil, bioretention would retain about 45 percent of the runoff generated and save about 40 percent of the pre-development recharge. To illustrate the relatively small role that slope increase from 5 to 10 percent plays in runoff retention, full retention would still be expected in the three residential cases and for the remaining two cases (COMM and REDEV) would decrease from 44-45 percent only slightly to 40-41 percent (not shown in table).

On the D soil, infiltrating bioretention may not be technically feasible and was not relied upon for retention estimates. Without the use of additional measures in the Full ARCD category, only incidental post-development runoff would be retained; and most pre-development recharge would be lost.

Tables 9-11 are companions to Table 8 for the South Central, Northeast – Upper Midwest, and Southwest climate regions, respectively. Results for the Northeast - Upper Midwest B soil are very close to those for the Southeast B soil, as would be expected given the similar precipitation quantities and soil characteristics. In the three regions having C soils, Basic ARCD can retain all runoff for the MFR, Sm-SFR, and Lg-SFR residential cases. With these soils, except in the Southwest, achieving full retention requires more of the available pervious area than with B soils, up to 69 percent, but is still fully attainable.

The effect of lower rainfall is evident in the South Central and, especially, the Southwest regions. In the latter location, not only the residential cases but also the COMM and REDEV scenarios can achieve full runoff retention with Basic ARCD on the C soil. The residential cases need much smaller percentages of the available pervious area for bioretention than for the same cases on C and even B soils elsewhere. Applying Basic ARCD to the South Central, C soil, REDEV case results in higher runoff retention than for the B soil cases in higher rainfall regions.

The study cases demonstrated two interesting points about groundwater recharge. First, with effective infiltrating bioretention it is possible for post-development annual recharge to exceed the pre-development quantity. This phenomenon is most evident in comparing the two amounts for cases with 100 percent runoff retention on C soils, which in the natural state produce much less recharge in relation to runoff than B soils. The B soils have a recharge-to-runoff ratio of about 500, whereas that ratio is only 4-6 for the C soils studied. One reason for higher post- compared to pre-development recharge is that bioretention is set up to hold water, increasing the time for infiltration to occur, instead of letting it run off. Another is that soils, especially in the C HSG, are often improved by organic amendments to yield both more water storage capacity and higher infiltration rates than the pre-existing soils.

A related point is that the percentage of pre-development recharge retained after development can be higher with C than B soils. This situation can best be seen in cases without full runoff retention, COMM and sometimes REDEV. In terms of recharge, installing bioretention conveys a greater advantage to the C than the B soils, which already have more pore space for water storage and higher infiltration and recharge rates.

Table 9. Runoff and Groundwater Recharge Volumes with Basic ARCD: South Central Climate Region[a]

Period	Volume (acre-ft) or Percentage Measure	MFR	Sm-SFR	Lg-SFR	COMM	REDEV
C soil						
Pre-dev.	Runoff	4.10	1.14	49.4	1.95	0.05
	Recharge	25.7	7.13	310	12.2	0.29
Post-dev.	Runoff without stormwater practices	21.2	5.15	224	12.7	0.31
	Runoff retained with Basic ARCD	21.2	5.15	224	4.33	0.21
	Runoff released with Basic ARCD	0	0	0	8.32	0.10
	Runoff retention (%)	100	100	100	34	67
	Recharge without stormwater practices	8.62	3.11	135	1.51	0.03
	Recharge with Basic ARCD	29.8	8.3	359	4.33	0.21
	Recharge retention (%)	100	100	100	38	70
	Pervious area needed (%)[b]	51	23	30	100	100
D soil						
Pre-dev.	Runoff	18.5	5.14	223	8.80	0.21
	Recharge	11.3	3.13	136	5.36	0.13
Post-dev.	Runoff without stormwater practices	Full ARCD needed to maximize retention on D soil				
	Runoff retained with Basic ARCD					
	Runoff released with Basic ARCD					
	Runoff retention (%)					
	Recharge without stormwater practices	7.23	7.59	112	1.35	0.03
	Recharge with Basic ARCD	Full ARCD needed to maximize retention on D soil				
	Recharge retention (%)	64	83	83	25	24
	Pervious area needed (%)[b]	Full ARCD needed to maximize retention on D soil				

Table 10. Runoff and Groundwater Recharge Volumes with Basic ARCD: Northeast – Upper Midwest Climate Region[a]

Period	Volume (acre-ft) or Percentage Measure	MFR	Sm-SFR	Lg-SFR	COMM	REDEV
B soil						
Pre-dev.	Runoff	0.04	0.01	0.54	0.02	0.001
	Recharge	42.9	11.9	517	20.4	0.49
Post-dev.	Runoff without stormwater practices	28.3	6.68	286	18.0	0.44
	Runoff retained with Basic ARCD	28.3	6.68	286	8.53	0.21
	Runoff released with Basic ARCD	0	0	0	9.43	0.23
	Runoff retention (%)	100	100	100	48	47
	Recharge without stormwater practices	14.6	5.32	231	2.42	0.06
	Recharge with Basic ARCD	42.9	11.9	517	8.53	0.21
	Recharge retention (%)	100	100	100	42	42
	Pervious area needed (%)[b]	34	21	21	100	100
C soil						
Pre-dev.	Runoff	7.87	2.18	94.8	3.74	0.09
	Recharge	35.1	9.72	422	16.6	0.40
Post-dev.	Runoff without stormwater practices	30.5	7.42	323	18.2	0.44
	Runoff retained with Basic ARCD	30.5	7.42	323	4.57	0.21
	Runoff released with Basic ARCD	0	0	0	13.6	0.24
	Runoff retention (%)	100	100	100	25	47
	Recharge without stormwater practices	12.4	4.48	195	2.17	0.05
	Recharge with Basic ARCD	42.9	11.9	517	4.57	0.21
	Recharge retention (%)	100	100	100	27	51
	Pervious area needed (%)[b]	69	31	40	100	100

Table 11. Runoff and Groundwater Recharge Volumes with Basic ARCD: Southwest Climate Region[a]

Period	Volume (acre-ft) or Percentage Measure	MFR	Sm-SFR	Lg-SFR	COMM	REDEV
C soil						
Pre-dev.	Runoff	1.62	0.45	19.5	0.77	0.02
	Recharge	7.22	2.00	87.0	3.43	0.08
Post-dev.	Runoff without stormwater practices	6.41	1.57	68.5	3.77	0.09
	Runoff retained with Basic ARCD	6.41	1.57	68.5	3.77	0.09
	Runoff released with Basic ARCD	0	0	0	0	0
	Runoff retention (%)	100	100	100	100	100
	Recharge without stormwater practices	2.43	0.88	38.1	0.43	0.01
	Recharge with Basic ARCD	8.84	2.45	107	4.20	0.10
	Recharge retention (%)	100	100	100	100	100
	Pervious area needed (%)[b]	12	5	7	69	44
D soil						
Pre-dev.	Runoff	4.47	1.24	53.8	2.12	0.05
	Recharge	4.37	1.21	52.7	2.08	0.05
Post-dev.	Runoff without stormwater practices	Full ARCD needed to maximize retention on D soil				
	Runoff retained with Basic ARCD					
	Runoff released with Basic ARCD					
	Runoff retention (%)					
	Recharge without stormwater practices	2.14	0.77	33.3	0.40	0.01
	Recharge with Basic ARCD	Full ARCD needed to maximize retention on D soil				
	Recharge retention (%)	49	63	63	19	18
	Pervious area needed (%)[b]	Full ARCD needed to maximize retention on D soil				

Full ARCD

Infiltration is one of a wide variety of ARCD-based source reduction techniques. Where site conditions such as soil quality or available area limit a site's infiltration capacity, other ARCD measures can enhance a site's runoff retention capability. Such practices can also be used where infiltration capacity is adequate, but the developer desires greater flexibility for land use on-site. Among those techniques, this study considered special management of roof water in those cases where bioretention could not infiltrate all post-development runoff.

Specifically, water harvesting for supply of irrigation and/or non-potable indoor uses was investigated for the retail commercial development. In residential cases with insufficient capacity for infiltrative bioretention but remaining space not already devoted to infiltration, efficiently directing roof runoff into the soil through downspout dispersion systems was the method of choice. Such cases invariably occurred with HSG D soils. The Full-ARCD scenario applied to the redevelopment case was roof water dispersion, harvesting, or a combination of the two practices. Generally speaking, infiltration consumed all available pervious area in the REDEV cases on B and C soils, making roof runoff harvesting the mechanism to retain more water. With no bioretention facility on D soil, the pervious area would be available for dispersion. Of course, harvesting could be applied instead of or along with dispersion. Again, it was assumed that that the commercial and, as needed, redevelopment sites had capacity to harvest and make use of the full volume of roof runoff generated, however, the analysis remains conservative in terms of the potential for onsite retention as it does not consider the use of ARCD site design principles to reduce impervious surfaces, green roofs, and evaporation/evapotranspration from surfaces other than rooftops.

Table 12 gives Southeast climate region results with the addition of Full ARCD techniques: roof runoff management, consisting of harvesting for reuse in the COMM case, dispersion on or within pervious land for the three residential cases, and a combination of these measures for REDEV. On the B soil runoff retention would approximately double for the retail commercial land use and reach 100 percent for the redevelopment. Groundwater recharge would not be expected to increase over the Basic ARCD case, though; because harvesting still keeps water out of the soil system.

For development on the D soil, use of roof runoff management techniques was estimated to increase runoff retention from zero to about one-third to two-thirds of the post-development runoff generated, depending on the land use case. Groundwater recharge would not materially benefit, however; because harvest does not contribute to it. Also, no recharge credit was taken for dispersion, since infiltration is restricted and loss by ET would tend to occur before infiltration. Some small amount of recharge would still be likely though. To illustrate further the small role of topography, in this D soil, Full ARCD scenario runoff retention is forecast to decrease by only 1-2 percent at a 10 percent slope compared to a 5 percent slope (not shown in table).

Table 12. Runoff and Groundwater Recharge Volumes with Full ARCD: Southeast Climate Region[a]

Period	Volume (acre-ft) or Percentage Measure	MFR	Sm-SFR	Lg-SFR	COMM	REDEV
B soil						
Pre-dev.	Runoff	0.046	0.013	0.56	0.022	0.001
	Recharge	44.7	12.4	539	21.2	0.51
Post-dev.	Runoff without stormwater practices	Complete retention possible with Basic ARCD			18.7	0.45
	Runoff retained with Full ARCD				16.1	0.45
	Runoff released with Full ARCD				2.66	0
	Runoff retention (%)				86%	100%
	Recharge without stormwater practices				2.53	0.06
	Recharge with Full ARCD				8.30	0.21
	Recharge retention (%)				39%	40%
	Pervious area needed (%)[b]				100%	100%
D soil						
Pre-dev.	Runoff	13.5	3.76	163	6.43	0.16
	Recharge	31.2	8.64	376	14.8	0.36
Post-dev.	Runoff without stormwater practices	33.1	8.23	358	19.1	0.46
	Runoff retained with Full ARCD	16.4	3.11	135	7.76	0.31
	Runoff released with Full ARCD	16.7	5.12	222	11.4	0.16
	Runoff retention (%)	50%	38%	38%	41%	66%
	Recharge without stormwater practices	11.6	4.17	181	2.12	0.05
	Recharge with Full ARCD	11.6	4.17	181	2.12	0.05
	Recharge retention (%)	37.2%	48.3%	48.3%	14.3%	13.6%
	Pervious area needed (%)[b]	100%	100%	100%	100%	100%

[a] Pre-dev.—pre-development; post-dev.—post-development; ARCD—aquatic resources conservation design; MFR—multi-family residential; Sm-SFR—small-scale single-family residential; Lg-SFR—large-scale single-family residential; COMM—retail commercial; REDEV—infill redevelopment; Full ARCD—infiltrating bioretention, roof runoff harvesting, and/or roof runoff dispersion; runoff—quantity of water discharged from the site on the surface; recharge--quantity of water infiltrating the soil
[b] Proportion of the total pervious area on the site required for bioretention to achieve given results

Tables 13-15 give data analogous to Table 12 for the South Central, Northeast – Upper Midwest, and Southwest climate regions, respectively. Results are similar to those reported for the Southeast region. Full ARCD can approximately double runoff retention from the Basic ARCD level for the COMM case and extend runoff retention to 100 percent for the redevelopment on both B and C soils. Once again, application of Full ARCD to the D soil cases increases runoff retention from zero to one-third to two-thirds of the volume produced, depending on land use case.

Table 13. Runoff and Groundwater Recharge Volumes with Full ARCD: South Central Climate Region[a]

Period	Volume (acre-ft) or Percentage Measure	MFR	Sm-SFR	Lg-SFR	COMM	REDEV
C soil						
Pre-dev.	Runoff	4.10	1.14	49.4	1.95	0.05
	Recharge	25.7	7.13	310	12.2	0.29
Post-dev.	Runoff without stormwater practices	Complete retention possible with Basic ARCD			12.7	0.31
	Runoff retained with Full ARCD				9.51	0.31
	Runoff released with Full ARCD				3.15	0
	Runoff retention (%)				75	100
	Recharge without stormwater practices				1.51	0.03
	Recharge with Full ARCD				4.33	0.21
	Recharge retention (%)				35	72
	Pervious area needed (%)[b]				100	100
D soil						
Pre-dev.	Runoff	18.5	5.14	223	8.80	0.21
	Recharge	11.3	3.13	136	5.36	0.13
Post-dev.	Runoff without stormwater practices	22.6	5.68	247	12.8	0.31
	Runoff retained with Full ARCD	11.0	2.08	90.3	5.17	0.20
	Runoff released with Full ARCD	11.6	3.60	157	7.63	0.11
	Runoff retention (%)	49	37	37	40	66
	Recharge without stormwater practices	7.23	2.59	112	1.35	0.03
	Recharge with Full ARCD	7.23	2.59	112	1.35	0.03
	Recharge retention (%)	64	83	83	25	24
	Pervious area needed (%)[b]	100	100	100	100	100

Table 14. Runoff and Groundwater Recharge Volumes with Full ARCD: Northeast – Upper Midwest Climate Region[a]

Period	Volume (acre-ft) or Percentage Measure	MFR	Sm-SFR	Lg-SFR	COMM	REDEV
B soil						
Pre-dev.	Runoff	0.04	0.01	0.54	0.02	0.001
	Recharge	42.9	11.9	51.7	20.4	0.49
Post-dev.	Runoff without stormwater practices	Complete retention possible with Basic ARCD			18.0	0.44
	Runoff retained with Full ARCD				16.0	0.44
	Runoff released with Full ARCD				2.00	0
	Runoff retention (%)				89	100
	Recharge without stormwater practices				2.42	0.06
	Recharge with Full ARCD				8.53	0.21
	Recharge retention (%)				42	43
	Pervious area needed (%)[b]				100	100
C soil						
Pre-dev.	Runoff	7.87	2.18	94.8	3.74	0.09
	Recharge	35.1	9.72	422	16.6	0.40
Post-dev.	Runoff without stormwater practices	Complete retention possible with Basic ARCD			18.2	0.44
	Runoff retained with Full ARCD				12.0	0.44
	Runoff released with Full ARCD				6.19	0
	Runoff retention (%)				66	100
	Recharge without stormwater practices				2.17	0.05
	Recharge with Full ARCD				4.57	0.21
	Recharge retention (%)				28	43
	Pervious area needed (%)[b]				100	100

Table 15. Runoff and Groundwater Recharge Volumes with Full ARCD: Southwest Climate Region[a]

Period	Volume (acre-ft) or Percentage Measure	MFR	Sm-SFR	Lg-SFR	COMM	REDEV
C soil						
Pre-dev.	Runoff	1.62	0.45	19.5	0.77	0.02
	Recharge	7.22	2.00	87.0	3.43	0.08
Post-dev.	Runoff without stormwater practices	Complete retention possible with Basic ARCD				
	Runoff retained with Full ARCD					
	Runoff released with Full ARCD					
	Runoff retention (%)					
	Recharge without *stormwater* practices					
	Recharge with Full ARCD					
	Recharge retention (%)					
	Pervious area needed (%)[b]					
D soil						
Pre-dev.	Runoff	4.47	1.24	53.8	2.12	0.05
	Recharge	4.37	1.21	52.7	2.08	0.05
Post-dev.	Runoff without stormwater practices	6.70	1.68	73.2	3.80	0.09
	Runoff retained with Full ARCD	3.25	0.62	26.8	1.53	0.06
	Runoff released with Full ARCD	3.45	1.07	46.5	2.26	0.03
	Runoff retention (%)	49	37	37	40	66
	Recharge without stormwater practices	2.14	0.77	33.3	0.40	0.01
	Recharge with Full ARCD	2.14	0.77	33.3	0.40	0.01
	Recharge retention (%)	49	63	63	19	18
	Pervious area needed (%)[b]	100	100	100	100	100

<u>Pollutant Loading Reductions</u>

The examination of maximum ARCD capabilities considered the reductions of annual mass loadings of four water pollutants that would accompany runoff retention. Since retention means no surface discharge, these loading reductions are, at a minimum, equal to the percentages of runoff retention. In those cases with less than full runoff retention, there is good reason to expect pollutant loading reductions higher than the percentage of runoff retained. The early runoff ("first flush"), occurring when the soils are least saturated, is more likely to be retained than later runoff. It is frequently observed that the first flush has higher pollutant concentrations than later runoff, particularly in the wash off after relatively extended dry periods.

For the B and D soil and the residential cases on C soils, the reductions were very consistent among regions:

- B and C soils, Basic ARCD, residential cases—100%;
- B soil, Basic ARCD, COMM and REDEV cases—44-45%;
- B soil, Full ARCD, COMM and REDEV cases—86-100%;
- D soil, Full ARCD, SFR and COMM cases—38-41%;
- D soil, Full ARCD, MFR case—50%; and
- D soil, Full ARCD, REDEV case—66%.

For the most highly impervious cases, COMM and REDEV, on C soils reduction was variable and dependent on precipitation. With Basic ARCD the range was from 25 to 100 percent, going from relatively high to low precipitation. Full ARCD is expected to raise the lowest reductions to 100 percent for REDEV and at least 66 percent for COMM.

Therefore, taking the greatest advantage of what ARCD offers could prevent the addition to receiving waters of all or almost all pollutant mass that would otherwise discharge from a range of urban developments on B and C soils. With D soils, Full ARCD can accomplish loading reductions approaching or somewhat exceeding 50 percent.

ABILITY TO MEET POTENTIAL STANDARDS

General Summary

This section evaluates the ability of the Basic and Full ARCD strategies to meet each of the five potential stormwater management standards enumerated in the beginning of the report. It also examines the extent of pollutant loading reduction if the standards are just met; i.e., if runoff is retained at the minimum needed to meet the standard. It has already been demonstrated that retention of all post-development runoff and full pollutant attenuation is possible in some circumstances. Table 16 summarizes the results for all regions and cases and both ARCD strategies.

Ability to Meet Standards

The projected ability to meet the standards overall varies mostly in relation to soil type (B or C versus D) and the relative imperviousness of development, and much less across climate regions. The one exception to this generality is that implementing Basic ARCD practices on the Southwest region C soil would meet all five standards. This uniformity does not occur elsewhere on either B or C soils, and is apparently primarily a function of the relatively low precipitation in the region.

Setting aside the Southwest region, success in complying with standards is mostly comparable among the various B and C soils, with a small number of instances where a development type meets a standard on B but not on C soil. Basic ARCD methods invariably can meet all standards on B and C soils for the residential development cases (MFR and Sm- and Lg-SFR). Full ARCD practices are forecast to meet all standards for the redevelopment case on B soils but only standards 1 and 5 consistently on C soils. The combination of infiltration and roof runoff management applied to the retail commercial development allows meeting these same two standards on B soils but only the latter on both of the C soils occurring outside the Southwest region. The only standards that cannot be met on B and C soils by the ARCD methods considered are standards 2-4 for the COMM case. Therefore, of the 125 standards assessments, ARCD practices are projected to meet 113 (90.4 percent) with B and C soils.

The ability to meet these standards is much reduced on D soils. Standard 1 can be met occasionally with Full ARCD used in the redevelopment. All cases with Full ARCD comply with standard 4 on this soil where pre-development runoff is estimated to be relatively high, reflecting a low overall requirement for retention volume. Standard 5 can be met with Full ARCD with the exception of one COMM case. Standards 2 and 3 were never estimated to be met in any D soil case. All in all, with this soil 26 of the 75 scenarios (34.7 percent) are expected to meet a standard.

Table 16. Ability to Meet Potential Regulatory Standards with Basic/Full ARCD Practices

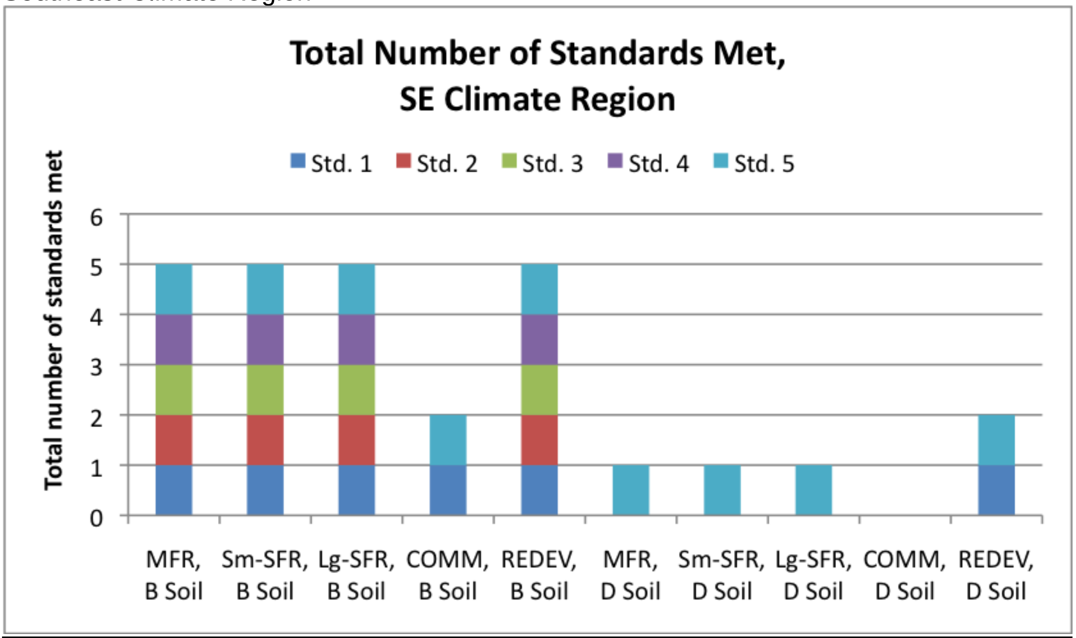
Region-Case[a]	Standards Met—Basic ARCD[b]	Standards Met—Full ARCD[b]	Runoff Retention and Pollutant Loading Reduction (%)[b, c]				
			Std. 1	Std. 2	Std. 3	Std. 4	Std. 5
SE(B)-MFR	1, 2, 3, 4, 5		63	87	90	>99	63
Sm-SFR	1, 2, 3, 4, 5		63	87	90	>99	63
Lg-SFR	1, 2, 3, 4, 5		63	87	90	>99	63
COMM		1, 5	63	86	86	86	63
REDEV		1, 2, 3, 4, 5	63	87	90	>99	63
SE(D)-MFR		5	50	50	50	50	37
Sm-SFR		5	38	38	38	38	34
Lg-SFR		5	38	38	38	38	34
COMM			41	41	41	41	41
REDEV		1, 5	63	66	66	66	42
SC(C)-MFR	1, 2, 3, 4, 5		58	82	90	81	47
Sm-SFR	1, 2, 3, 4, 5		58	82	90	78	45
Lg-SFR	1, 2, 3, 4, 5		58	82	90	78	45
COMM		1, 5	58	75	75	75	49
REDEV		1, 2, 3, 4, 5	58	82	90	84	49
SC(D)-MFR		4, 5	49	49	49	18	10
Sm-SFR		4, 5	37	37	37	10	6
Lg-SFR		4, 5	37	37	37	10	6
COMM		4, 5	40	40	40	31	18
REDEV		1, 4, 5	58	66	66	32	18
NM(B)-MFR	1, 2, 3, 4, 5		81	89	90	>99	81
Sm-SFR	1, 2, 3, 4, 5		81	89	90	>99	81
Lg-SFR	1, 2, 3, 4, 5		81	89	90	>99	81
COMM		1, 2, 5	81	89	89	89	81
REDEV		1, 2, 3, 4, 5	81	89	90	>99	81
NM(C)-MFR	1, 2, 3, 4, 5		81	89	90	74	60
Sm-SFR	1, 2, 3, 4, 5		81	89	90	71	57
Lg-SFR	1, 2, 3, 4, 5		81	89	90	71	57
COMM		5	66	66	66	66	64
REDEV		1, 2, 3, 4, 5	81	89	90	80	64
SW(C)-MFR	1, 2, 3, 4, 5		62	83	90	75	46
Sm-SFR	1, 2, 3, 4, 5		62	83	90	72	44
Lg-SFR	1, 2, 3, 4, 5		62	83	90	72	44
COMM	1, 2, 3, 4, 5		62	83	90	80	49
REDEV	1, 2, 3, 4, 5		62	83	90	80	49
SW(D)-MFR		4, 5	49	49	49	33	21
Sm-SFR		4, 5	37	37	37	27	16
Lg-SFR		4, 5	37	37	37	27	16
COMM		5	40	40	40	40	27
REDEV		1, 4, 5	62	66	66	44	28

[a] Region (hydrologic soil group)—land use; regions: SE—Southeast, SC—South-central, NM—Northeast-Upper Midwest, SW—Southwest; land uses: MFR—multi-family residential, Sm-SFR—small single-family residential, Lg-SFR--large single-family residential, COMM—retail commercial, REDEV--redevelopment

[b] Standard (Std.) 1—Retain the runoff produced by the 85th percentile, 24-hour precipitation event
Standard 2—Retain the runoff produced by the 95th percentile, 24-hour precipitation event
Standard 3—Retain 90 percent of the average annual post-development runoff volume
Standard 4—Retain the difference between the post- and pre-development average annual runoff volumes
Standard 5—Retain the difference between the post- and pre-development runoff volumes for all events up to and including the 85th percentile, 24-hour precipitation event

[c] Reduction estimated to result from meeting the standard, to the extent it can be met (fully met if so indicated in preceding columns), without treatment of remaining discharge. Where a standard can be met using Basic or Full ARCD application it is indicated in black, where a standard cannot be met using Basic or Full ARCD it is highlighted red.

Figure 4a. Ability to Meet Potential Regulatory Standards with Basic/Full ARCD Practices for Southeast Climate Region



MFR—multi-family residential, Sm-SFR—small single-family residential, Lg-SFR--large single-family residential, COMM—retail commercial, REDEV—redevelopment. Standard (Std.) 1—Retain the runoff produced by the 85th percentile, 24-hour precipitation event; Standard 2—the 95th percentile, 24-hour precipitation event; Standard 3—90 percent of the average annual post-development runoff volume; Standard 4—the difference between the post- and pre-development average annual runoff volumes; and, Standard 5—the difference between the post- and pre-development runoff volumes for all events up to and including the 85th percentile, 24-hour precipitation event

Figure 4b. Ability to Meet Potential Regulatory Standards with Basic/Full ARCD Practices for South Central Climate Region

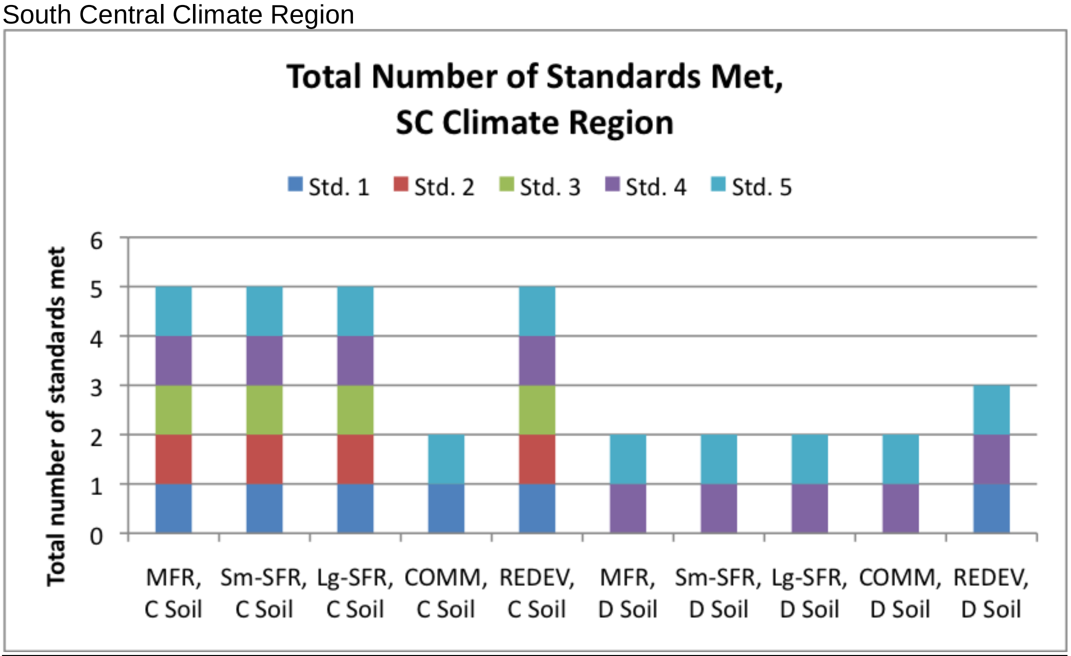


Figure 4c. Ability to Meet Potential Regulatory Standards with Basic/Full ARCD Practices for Northeast-Midwest Climate Region

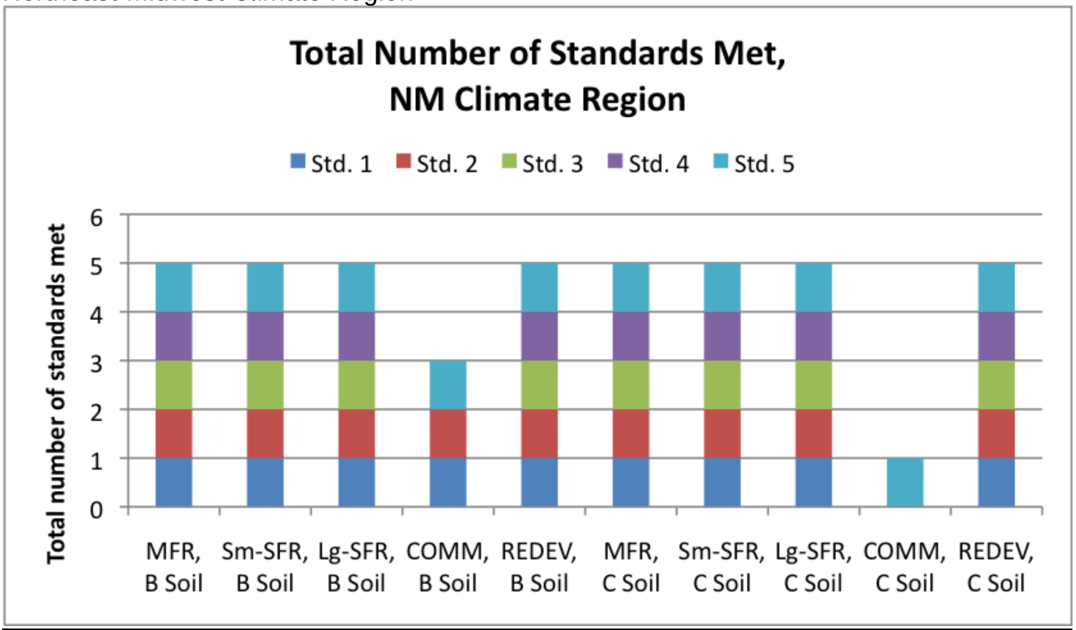


Figure 4d. Ability to Meet Potential Regulatory Standards with Basic/Full ARCD Practices for Southwest Climate Region

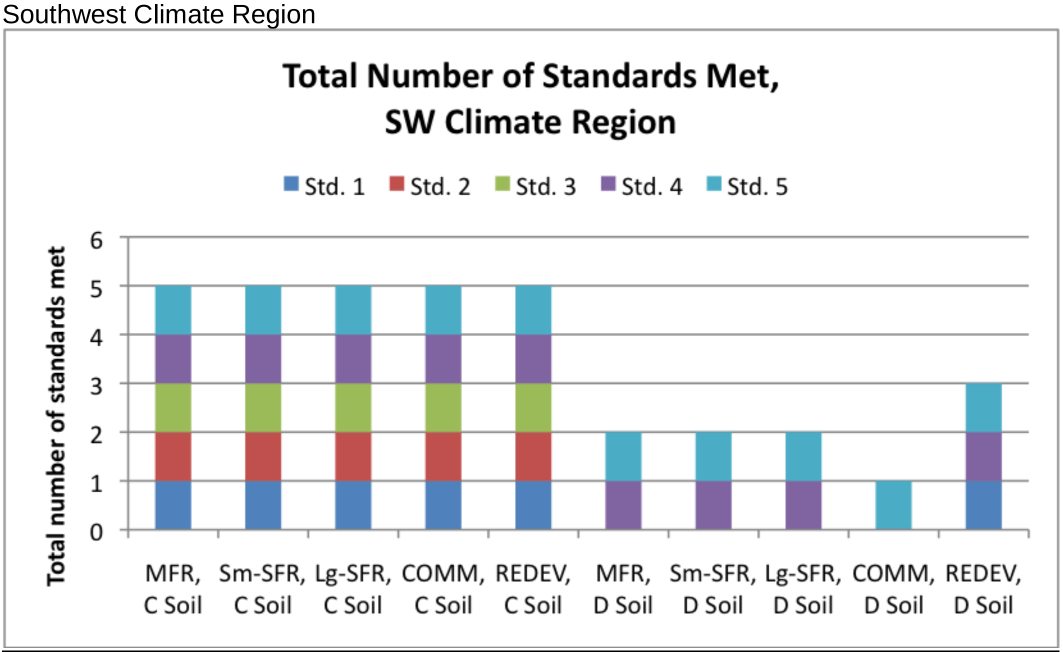
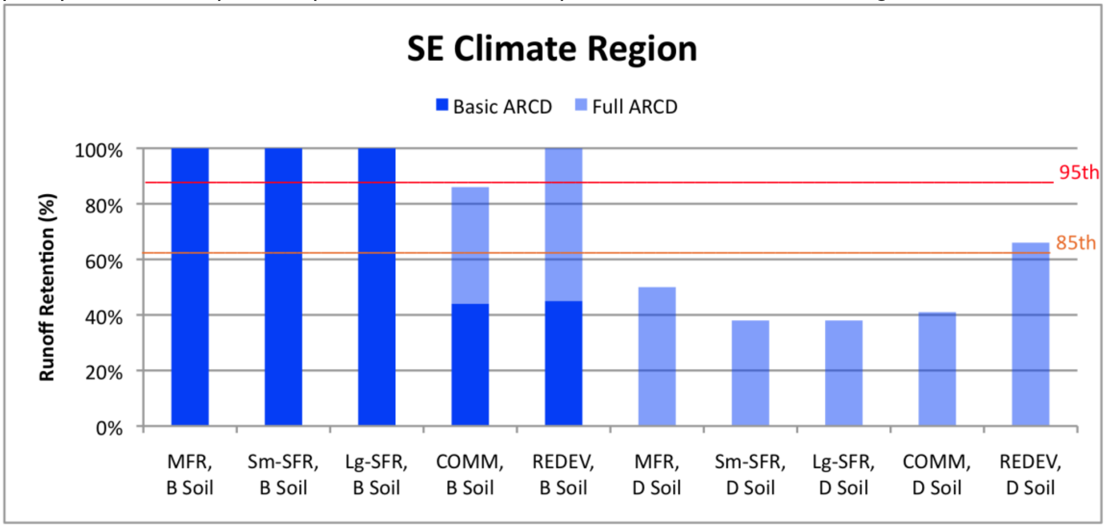


Figure 5a. Percentage of Runoff Retained Relative to Standards 1 (85th Percentile, 24-hour precipitation event) and 2 (95th Percentile event) for Southeast Climate Region



MFR—multi-family residential, Sm-SFR—small single-family residential, Lg-SFR--large single-family residential, COMM—retail commercial, REDEV—redevelopment. Standard (Std.) 1—Retain the runoff produced by the 85th percentile, 24-hour precipitation event; Standard 2—the 95th percentile, 24-hour precipitation event; Standard 3—90 percent of the average annual post-development runoff volume; Standard 4—the difference between the post- and pre-development average annual runoff volumes; and, Standard 5—the difference between the post- and pre-development runoff volumes for all events up to and including the 85th percentile, 24-hour precipitation event

Figures 5a-d show the percentage of runoff that can be retained for each development type, in each region, using either Basic or Full ARCD practices, in comparison with Standard 1 (retention of the 85th percentile, 24-hour precipitation event) and Standard 2 (retention of the 95th percentile, 24 hour event). Even where Standards 1 and 2 cannot be met in full, ARCD practices can still result in substantial compliance, and retention of significant runoff volume.

Figure 5b. Percentage of Runoff Retained Relative to Standards 1 (85th Percentile, 24-hour precipitation event) and 2 (95th Percentile event) for South Central Climate Region

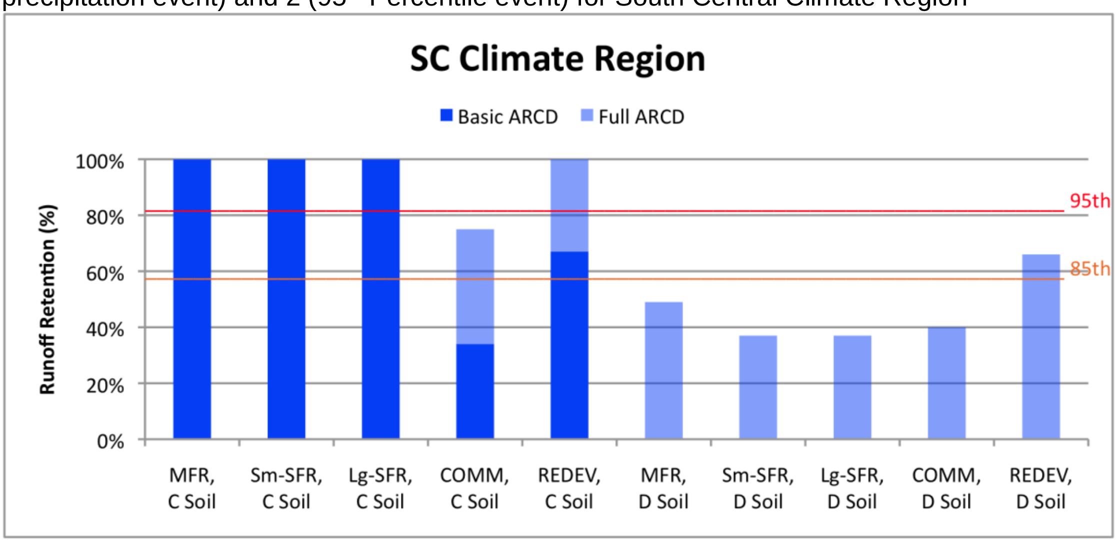


Figure 5c. Percentage of Runoff Retained Relative to Standards 1 (85th Percentile, 24-hour precipitation event) and 2 (95th Percentile event) for Northeast-Midwest Region

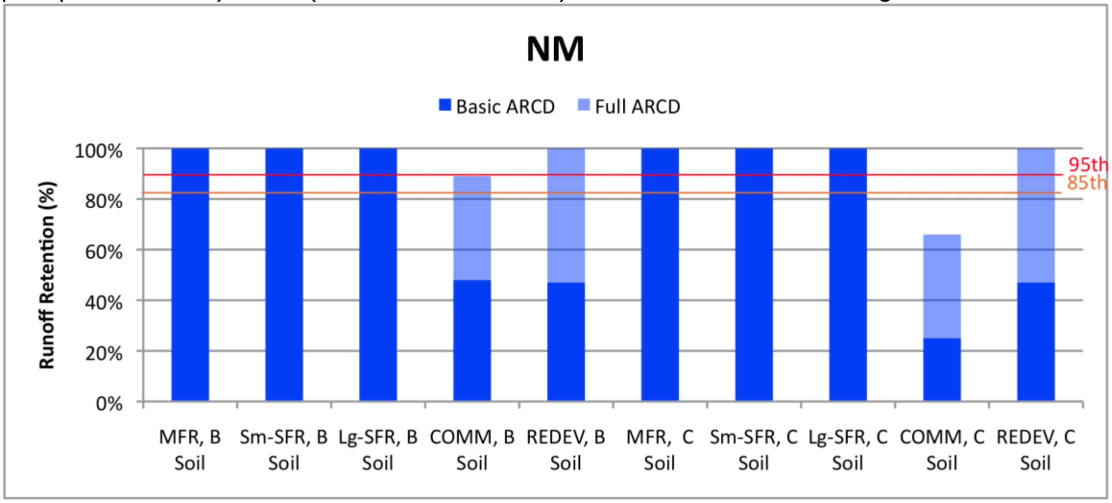
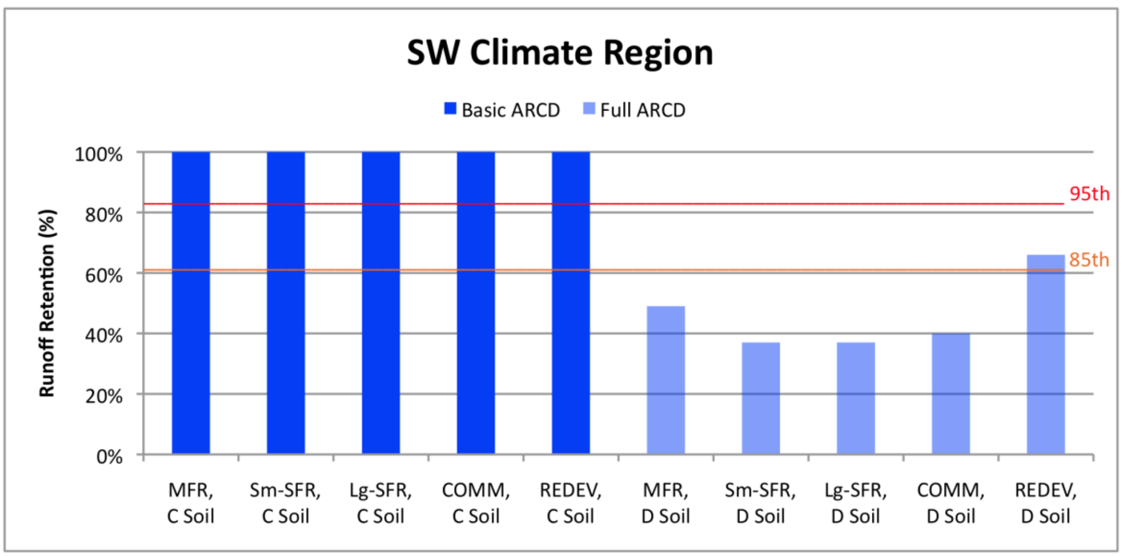


Figure 5d. Percentage of Runoff Retained Relative to Standards 1 (85th Percentile, 24-hour precipitation event) and 2 (95th Percentile event) for Southwest Region



<u>Effectiveness of Standards in Environmental Protection</u>

Standard 3 (retain 90 percent of the average annual post-development runoff volume) would be the most protective standard. Meeting or coming as close as possible to meeting, but not exceeding, this standard is estimated to lead to 66-90 percent runoff retention and pollutant loading reduction on B and C soils and 37-66 percent on D soil. Standard 2 (retain the runoff produced by the 95th percentile, 24-hour precipitation event) would yield only slightly less protection with B and C soils and, with D soil, retention and loading reduction equivalent to standard 3.

Standards 4 and 5, based on the differential between pre- and post-development runoff volume, are highly inconsistent in retaining runoff and reducing pollutants, in that they are relatively protective where pre-development runoff is estimated to be very low relative to post-development flow, but result in progressively lower retention and pollutant loading reduction as pre- and post-development volumes converge, such as in several cases on D soils. Standard 5 is especially weak in this regard. The potentially low level of retention and pollutant loading reduction renders these standards based on the change in pre- versus post-development runoff volume poor candidates for national application, at least as formulated in these terms.

Fully meeting standard 1 (retain the runoff produced by the 85th percentile, 24-hour precipitation event) would yield runoff retention and pollutant mass reduction ranging from 58 to 81 percent, depending on climate region. This level of inconsistency decreases the utility of this standard for widespread use. Standard 2, based on the 95th percentile event, is much better in this respect, with variability in runoff retention and loading reduction across the nation in the much narrower 82-89 percent range. However, standard 1 remains more consistent across regions, and more protective of water quality for development on D soils than either standard 4 or 5, and is preferable to those standards in this regard.

In summary, standards 2 and 3 are clearly superior to the other three options. Standard 3 is entirely consistent from place to place in degree of environmental protection, and standard 2 does not deviate much. Analysis of the five development cases on two soil groups in each of four regions demonstrated the two standards are virtually identical in the runoff retention and pollutant loading reduction they would bring about.

<u>Management or Runoff in Excess of Standards Requirements</u>

All of the analysis reported above assumed that any remaining runoff after the application of ARCD and meeting, or coming as close as possible to meeting a standard, would discharge with no treatment. In fact, additional treatment could further decrease pollutant loadings. Treatment without further runoff retention could be accomplished by many conventional or ARCD methods designed to lower contaminant concentrations. The most effective of the alternatives is probably bioretention discharging non-retained runoff either on the surface or through an underdrain, assumed in the analysis conducted for this study according to the methods cited above. Treatment of all remaining runoff with underdrained bioretention cells where space remains but all infiltration capacity is used can raise the pollutant removals given in Table 16 to the levels in Table 17. These estimates apply to the four pollutants considered, TSS and total copper, zinc, and phosphorus. Space would most likely be available in the three MFR and SFR cases but not the COMM and REDEV scenarios.

While there is substantial variability in these results, they demonstrate that discharging effluent of relatively consistent, high quality can be accomplished with a comprehensive ARCD strategy. This strategy would embrace, first, retaining as much urban runoff as possible and then utilizing treatment based on soil and vegetative media to capture contaminants from the remainder.

Table 17. Estimated Pollutant Loading Reduction Benefits of Bioretention Treatment of Runoff Remaining After ARCD Implemented to Meet or Approach Standards

Range of Table 16 Values (%)	Approximate Pollutant Removal Increase (%)	Total Estimated Pollutant Removal Range (%)
35-45	30-45	65-90
45-55	25-35	70-90
55-65	20-30	75-95
65-75	15->20	80->95
75-85	10->15	85->95
>85	5->10	90->95

SUMMARY AND CONCLUSIONS

STUDY DESIGN

This study was performed to investigate the degree to which low-impact development ARCD practices can meet or exceed the requirements of various potential stormwater management facility design standards and the resulting environmental benefits. The investigation was performed by estimating the stormwater retention possible with full application of ARCD practices to five land use cases in four representative climatic regions in the United States on two prominent soil types in each region. Retention is defined as preventing the conversion of precipitation to surface runoff. Retaining runoff from impervious and pollutant generating pervious surfaces prevents the introduction of urban runoff pollutants to receiving waters as well as reduces runoff volume to prevent stream channel and habitat damage, flooding, and loss of groundwater recharge. Infiltrating bioretention was first applied in the analysis of each case, a strategy termed Basic ARCD. When Basic ARCD could not fully retain post-development runoff, a Full ARCD strategy was added, involving roof runoff harvesting in the most impervious development cases and roof water dispersion in those with substantial pervious area. Benefits were assessed with respect to reduction of the annual average surface runoff volume from the quantity estimated without any stormwater management practices, and associated maintenance of pre-development groundwater recharge and water quality improvement through preventing discharge to receiving waters of pollutants generated with developed land uses.

A number of conservative assumptions were built into the analysis to ensure that the capabilities and benefits of ARCD would not be over-estimated. In summary, these assumptions are:

- No retention credit for evapotranspiration in the Basic ARCD strategy, although generally a substantial amount would occur, and consideration of evapotranspiration only for roof runoff in the Full ARCD strategy;
- Letting aside many available ARCD practices and site design principles that could be employed to reduce the runoff quantity, and the pollutants it transports, by reducing impervious surface area or directing the runoff to bioretention, harvesting, and dispersion facilities;
- The assumption of no infiltration on hydrologic soil group D soils, although some infiltration occurs at finite rates even on clay;
- Application of a safety factor to estimated infiltration rates;
- Minimum bioretention cell depths, so that these facilities would not be disruptive to site design and could be put to other uses;
- Requiring a 48-hour drawdown time for bioretention, instead of the 72-hour maximum;
- An analysis to guard against groundwater mounding under bioretention cells, with conservative assumptions for horizontal and vertical hydraulic conductivity rates; and
- An analysis demonstrating that doubling topographic slope changes results by only a few percent.

CAPABILITIES OF FULL ARCD APPLICATION

Comparison of estimated runoff production in the pre- and post-development states demonstrated that the majority of the infiltration that would recharge groundwater in the undeveloped state would be lost to surface runoff after development with no stormwater management practices. These losses would approach 90 percent in the most impervious developments. These observations apply in in all climate regions and with the full range of soil conditions.

Basic ARCD could retain all post-development runoff and pre-existing groundwater recharge, as well as attenuate all pollutant transport, in the three residential cases on B soils in the two climate regions where these soils were analyzed. Bioretention cells to accomplish this retention would use from less than one-fourth to just over one-third of the available pervious area for infiltration. Taking all available pervious area for the more highly impervious COMM and REDEV cases, bioretention would retain about 45 percent of the runoff and pollutants generated and save about 40 percent of the pre-development recharge. Adding Full ARCD measures in these cases would approximately double retention and pollutant reduction for the retail commercial land use and raise it to 100 percent for the redevelopment. Groundwater recharge would not increase, however, because the additional retention is accomplished by harvesting or dispersion.

In the three regions having C soils, Basic ARCD can again retain all runoff and reduce urban runoff pollutant mass loading to zero for the MFR and Sm-SFR and Lg-SFR residential cases, although generally requiring more of the available pervious area to do so than in B soil cases. The effect of lower rainfall is evident in the South Central and, especially, the Southwest regions. In the latter location, not only the residential cases but also the COMM and REDEV scenarios can achieve full runoff and groundwater recharge retention and pollutant loading attenuation with Basic ARCD on C soil. Full ARCD can approximately double runoff retention and pollutant removal from the Basic ARCD level for the COMM case and extend these measures to 100 percent for the redevelopment.

For development on the D soils in all climate regions, use of roof runoff management techniques was estimated to increase runoff retention and pollutant reduction from zero to between about one-third to two-thirds of the post-development runoff generated, depending on the land use case. These strategies would offer little groundwater recharge benefit with this soil condition, but would still have the potential to significantly reduce runoff volume and pollutant loading.

Therefore, taking the greatest advantage of what ARCD offers is expected to retain the great majority of post-development runoff and pre-development groundwater recharge. This strategy would also prevent the addition to receiving waters of all or almost all pollutant mass that would otherwise discharge from a range of urban developments on B and C soils. With D soils, Full ARCD can accomplish runoff retention and loading reductions approaching or somewhat exceeding 50 percent, and opportunities to use ARCD practices or site design principles not modeled in this analysis can further increase runoff retention volume.

ABILITY TO MEET STANDARDS

ARCD methods were assessed for their ability to meet five potential regulatory standards, the first two pertaining to retention of the 85th and 95th percentile, 24-hour precipitation events, the third to retain 90 percent of the post-development runoff, and the last two to retain the difference between the post- and pre-development runoff, the final standard capped at the 85th percentile, 24-hour event. The projected ability to meet the five standards varies mostly in relation to soil type (B or C versus D) and the relative imperviousness of development, and much less across climate regions, except for the relatively arid Southwest.

The only standards that cannot be fully met on B and C soils by the ARCD methods considered are standards 2-4 for the COMM case. Of the 125 standards assessments, ARCD practices are projected to meet 113 (90.4 percent) with B and C soils. The ability to meet these standards is much reduced on D soils. Only standards 1 (85^{th} percentile, 24-hour precipitation event, and 4 and 5 (related to the difference between the post- and pre-development runoff) can be met occasionally and under limited conditions using Full ARCD methods. However, even on D soils, all cases for Standard 1 were able to retain greater than 50 percent of the required runoff volume.

Standard 3 (retain 90 percent of the average annual post-development runoff volume) would be the most environmentally protective standard. Meeting or coming as close as possible to meeting, but not exceeding, this standard was estimated to lead to 66-90 percent runoff retention and pollutant loading reduction on B and C soils and 37-66 percent on D soil. Standard 2 (retain the runoff produced by the 95^{th} percentile, 24-hour precipitation event) would yield equivalent protection on D soils and only slightly less protection with B and C soils.

Standards 4 and 5, based on the differential between pre- and post-development runoff volume, are very inconsistent in retaining runoff and reducing pollutants. They are highly protective where pre-development runoff is estimated to be very low relative to post-development flow, and then to result in progressively lower retention and loading reduction as pre- and post-development volumes converge. Standard 5 is especially weak in this regard. This inconsistency makes these standards poor candidates for national application, at least as formulated in these terms.

Fully meeting standard 1 (retain the runoff produced by the 85^{th} percentile, 24-hour precipitation event) would yield runoff retention and pollutant mass reduction ranging from 58 to 81 percent, depending on climate region. This level of inconsistency decreases the utility of this standard to some degree. Standard 2, based on the 95^{th} percentile event, is much better in this respect, with variability in runoff retention and loading reduction across the nation in the much narrower 82-89 percent range. However, standard 1 remains more consistent across regions, and more protective of water quality for development on D soils than either standard 4 or 5, and is preferable to those standards in this regard.

In summary, standards 2 and 3 are clearly superior to the other three options. Standard 3 is entirely consistent from place to place in degree of environmental protection, and standard 2 does not deviate much. Analysis of the five development cases on two soil groups in each of four regions demonstrated the two standards are virtually identical in the runoff retention and pollutant loading reduction they would bring about.

All five standards are based on some stipulated runoff retention. Pollutant mass loading reduction is at least equal to the amount of retention that occurs. It is possible to decrease loadings further by treating excess runoff. Analysis showed that subjecting that runoff to bioretention treatment before discharge could reduce loadings of TSS and total copper, zinc, and phosphorus by at least two-thirds and as much as over 95 percent. This conclusion applies to all climate regions and soil types for land use cases where space is available for the additional bioretention cells. The three residential cases are in this group but not the COMM or REDEV cases, where all pervious land would have already been used for retentive or roof water dispersion practices.

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ATTACHMENT A

POLLUTANT CONCENTRATIONS FOR URBAN SOURCE AREAS (HERRERA ENVIRONMENTAL CONSULTANTS, INC. UNDATED)

Source Area	Study	Location	Sample Size (n)	TSS (mg/L)	TCu (µg/L)	TPb (µg/L)	TZn (µg/L)	TP (mg/L)	Notes
Roofs									
Residential	Steuer, et al. 1997	MI	12	36	7	25	201	0.06	2
Residential	Bannerman, et al. 1993	WI	~48	27	15	21	149	0.15	3
Residential	Waschbusch, et al. 2000	WI	25	15	n.a.	n.a.	n.a.	0.07	3
Residential	FAR 2003	NY		*19*	*20*	*21*	*312*	*0.11*	4
Residential	Gromaire, et al. 2001	France		*29*	*37*	*493*	*3422*	*n.a.*	5
Representative Residential Roof Values				**25**	**13**	**22**	**159**	**0.11**	
Commercial	Steuer, et al. 1997	MI	12	24	20	48	215	0.09	2
Commercial	Bannerman, et al. 1993	WI	~16	15	9	9	330	0.20	3
Commercial	Waschbusch, et al. 2000	WI	25	18	n.a.	n.a.	n.a.	0.13	3
Representative Commercial Roof Values				**18**	**14**	**26**	**281**	**0.14**	
Parking Areas									
Res. Driveways	Steuer, et al. 1997	MI	12	157	34	52	148	0.35	2
Res. Driveways	Bannerman, et al. 1993	WI	~32	173	17	17	107	1.16	3
Res. Driveways	Waschbusch, et al. 2000	WI	25	34	n.a.	n.a.	n.a.	0.18	3
Driveway	FAR 2003	NY		*173*	*17*		*107*	*0.56*	4
Representative Residential Driveway Values				**120**	**22**	**27**	**118**	**0.66**	
Comm./ Inst. Park. Areas	Pitt, et al. 1995	AL	16	110	116	46	110	n.a.	1
Comm. Park. Areas	Steuer, et al. 1997	MI	12	110	22	40	178	0.2	2
Com. Park. Lot	Bannerman, et al. 1993	WI	5	58	15	22	178	0.19	3
Parking Lot	Waschbusch, et al. 2000	WI	25	51	n.a.	n.a.	n.a.	0.1	3
Parking Lot	Tiefenthaler, et al. 2001	CA	5	36	28	45	293	n.a.	6
Loading Docks	Pitt, et al. 1995	AL	3	40	22	55	55	n.a.	1
Highway Rest Areas	CalTrans 2003	CA	53	*63*	*16*	*8*	*142*	*0.47*	7

Park and Ride Facilities	CalTrans 2003	CA	179	*69*	*17*	*10*	*154*	*0.33*	7
Comm./ Res. Parking	FAR 2003	NY		*27*	*51*	*28*	*139*	*0.15*	4
Representative Parking Area/Lot Values				**75**	**36**	**26**	**97**	**0.14**	
Landscaping/Lawns									
Landscaped Areas	Pitt, et al. 1995	AL	6	33	81	24	230	n.a.	1
Landscaping	FAR 2003	NY		*37*	*94*	*29*	*263*	n.a.	4
Representative Landscaping Values				**33**	**81**	**24**	**230**	**n.a.**	
Lawns - Residential	Steuer, et al. 1997	MI	12	262	n.a.	n.a.	n.a.	2.33	2
Lawns - Residential	Bannerman, et al. 1993	WI	~30	397	13	n.a.	59	2.67	3
Lawns	Waschbusch, et al. 2000	WI	25	59	n.a.	n.a.	n.a.	0.79	3
Lawns	Waschbusch, et al. 2000	WI	25	122	n.a.	n.a.	n.a.	1.61	3
Lawns - Fertilized	USGS 2002	WI	58	n.a.	n.a.	n.a.	n.a.	2.57	3
Lawns - Non-P Fertilized	USGS 2002	WI	38	n.a.	n.a.	n.a.	n.a.	1.89	3
Lawns - Unfertilized	USGS 2002	WI	19	n.a.	n.a.	n.a.	n.a.	1.73	3
Lawns	FAR 2003	NY	3	*602*	*17*	*17*	*50*	*2.1*	4
Representative Lawn Values				**213**	**13**	**n.a.**	**59**	**2.04**	

Notes:

Representative values are weighted means of collected data. Italicized values were omitted from these calculations.

1 - Grab samples from residential, commercial/institutional, and industrial rooftops. Values represent mean of DETECTED concentrations

2 - Flow-weighted composite samples, geometric mean concentrations

3 - Geometric mean concentrations

4 - Citation appears to be erroneous - original source of data is unknown. Not used to calculate representative value

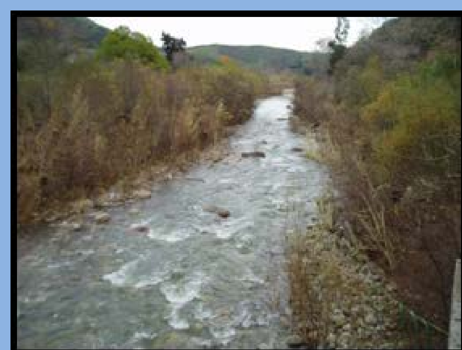
5 - Median concentrations. Not used to calculate representative values due to site location and variation from other values.

6 - Mean concentrations from simulated rainfall study

7 - Mean concentrations. Not used to calculate representative values due to transportation nature of land use.

Ventura County Technical Guidance Manual for Stormwater Quality Control Measures

Manual Update 2011



Ventura Countywide Stormwater Quality Management Program

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Manual Updates: The 2011 TGM may be periodically updated to correct minor errors and unintentional omissions. Additionally, due to the evolving nature of stormwater quality management, the 2011 TGM may also be updated to incorporate new and innovative control measures. 2011 TGM users should ensure that they are referencing the most current edition by checking www.vcstormwater.org or contacting the local permitting agency.

TABLE OF CONTENTS

1 INTRODUCTION........1-1

1.1 Goals........1-1
1.2 Regulatory Background........1-2
1.3 Impacts of Land Development........1-4
1.4 Stormwater Management Principles........1-5
1.5 Applicability........1-6
New Development Projects........1-6
Redevelopment Projects........1-7
Effective Date........1-8
1.6 Organization of the 2011 TGM........1-9

2 STORMWATER MANAGEMENT STANDARDS........2-1

2.1 Introduction........2-1
2.2 Step 1: Determine Project Applicability........2-1
Step 1a: Determine RPAMP Eligibility........2-1
Step 1b: Single-Family Hillside Homes........2-3
Step 1c: Roadway Projects........2-6
2.3 Step 2: Assess Site Conditions........2-7
2.4 Step 3: Apply Site Design Principles and Techniques........2-7
2.5 Step 4: Apply Source Control Measures........2-8
2.6 Step 5: Apply BMPs to Reduce EIA to ≤5%........2-9
Step 5a: Calculate Allowable EIA........2-9
Step 5b: Calculate Impervious Area to be Retained........2-11
Step 5c: Calculate the Volume to be Retained (SQDV)........2-12
Step 5d: Select and Size Onsite Retention BMPs to Achieve 5% EIA........2-14
Step 5e: Select and Size Biofiltration BMPs to Reduce EIA to ≤5%........2-14
2.7 Step 6: Alternative Compliance........2-16
2.8 Step 7: Apply Treatment Control Measures........2-28
2.9 Step 8: Continue Project Design Process: Flood Control and Hydromodification Requirements........2-31
Step 8a: Flood Control Requirements........2-31
Step 8b: Hydromodification (Flow/Volume/Duration) Control Criteria........2-31
2.10 Step 9: Develop Maintenance Plan........2-33

3 SITE ASSESSMENT AND BMP SELECTION........3-1

3.1 Assessing Site Conditions and Other Constraints........3-1
Site Conditions........3-1
3.2 Technical Feasibility Screening........3-6
Determining Maximum Volume Feasibly Infiltrated and/or Biofiltered........3-10

3.3 Treatment Control Measure Selection Guidance 3-13
Primary Class of Pollutants 3-13
Consideration of Site-Specific Conditions 3-18

4 SITE DESIGN PRINCIPLES AND TECHNIQUES 4-1

4.1 Introduction 4-1
4.2 Site Planning 4-2
Purpose 4-2
Design Criteria 4-2
4.3 Protect and Restore Natural Areas 4-4
Purpose 4-4
Design Criteria 4-4
4.4 Minimize Land Disturbance 4-6
Purpose 4-6
Design Criteria 4-6
4.5 Minimize Impervious Cover 4-7
Purpose 4-7
Design Criteria 4-7
4.6 Apply LID at Various Scales 4-9
Purpose 4-9
Design Criteria 4-9
4.7 Implement Integrated Water Resource Management Practices 4-11
Purpose 4-11
Design Criteria 4-11

5 SOURCE CONTROL MEASURES 5-1

5.1 Introduction 5-1
5.2 Description 5-1
5.3 Site-Specific Source Control Measures 5-3
S-1: Storm Drain Message and Signage 5-3
S-2: Outdoor Material Storage Area Design 5-5
S-3: Outdoor Trash Storage Area Design 5-6
S-4: Outdoor Loading/Unloading Dock Area Design 5-7
S-5: Outdoor Repair/Maintenance Bay Design 5-9
S-6: Outdoor Vehicle/Equipment/Accessory Washing Area Design 5-10
S-7: Fueling Area Design 5-11
S-8: Proof of Control Measure Maintenance 5-12

6 STORMWATER BMP DESIGN 6-1

6.1 Introduction 6-1
6.2 General Considerations 6-1
Maintenance Responsibility 6-2

Pretreatment 6-2
Infiltration 6-3
Biofiltration BMPs 6-4
Treatment Control Measures 6-5
"On-line" and "Off-line" Facilities 6-5
6.3 Retention BMP, Biofiltration BMP, and Treatment Control Measure Fact Sheets 6-6
INF-1: Infiltration Basin 6-8
INF-2: Infiltration Trench 6-24
INF-3: Bioretention 6-33
INF-4: Drywell 6-49
INF-5: Permeable Pavement 6-57
INF-6: Proprietary Infiltration 6-72
INF-7: Bioinfiltration 6-78
RWH-1: Rainwater Harvesting 6-94
ET-1: Green Roof 6-102
ET-2: Hydrologic Source Control BMPs 6-108
BIO-1: Bioretention with Underdrain 6-116
BIO-2: Planter Box 6-129
BIO-3: Vegetated Swale 6-139
BIO-4: Vegetated Filter Strip 6-154
BIO-5: Proprietary Biotreatment 6-163
TCM-1: Dry Extended Detention Basin 6-166
TCM-2: Wet Detention Basin 6-187
TCM-3: Constructed Wetland 6-207
TCM-4: Sand Filters 6-224
TCM-5: Cartridge Media Filter 6-236
PT-1: Hydrodynamic Separation Device 6-240
PT-2: Catch Basin Insert 6-244

7 MAINTENANCE PLAN 7-1

7.1 Site Map 7-1
7.2 Baseline Descriptions 7-1
7.3 Spill Plan 7-2
7.4 Facility Changes 7-2
7.5 Training 7-3
7.6 Basic Inspection and Maintenance Activities 7-3
7.7 Revisions of Pollution Mitigation Measures 7-3
7.8 Monitoring & Reporting Program 7-4

LIST OF TABLES

Table 2-1: Flow-Based Biofiltration BMP Design Intensity for 150% Sizing 2-15
Table 2-2: Allowed Design Storm Methodology Based on Project Size 2-29
Table 2-3: Ventura Soil Type Pervious Runoff Coefficients 2-30
Table 3-1: Recommended Criteria for Percent of Site Feasible to Dedicate to BMPs....... 3-12
Table 3-2: Land Uses and Associated Pollutants 3-14
Table 3-3: Ventura County 2010 303(d)-listed Water Quality Pollutants 3-15
Table 3-4: Treatment Control Measures for Addressing Pollutants of Concern 3-16
Table 3-5: BMP Site Suitability Considerations.. 3-19
Table 3-6: BMP Cost Considerations 3-21
Table 4-1: Rule of Thumb Space Requirements for BMPs 4-3
Table 5-1: Summary of Site-Specific Source Control Measure Design Features 5-2
Table 5-2: Design Criteria for Outdoor Material Storage Area Design 5-6
Table 5-3: Design Criteria for Outdoor Trash Storage Areas 5-7
Table 5-4: Design Criteria for Outdoor Loading/ Unloading Areas 5-8
Table 6-1: Infiltration Basin Design Criteria 6-11
Table 6-2: Suitability Assessment Related Considerations for Infiltration Facility Safety Factors 6-15
Table 6-3: Design Related Considerations for Infiltration Facility Safety Factors 6-17
Table 6-4: Infiltration Facility Safety Factor Determination Worksheet.............................. 6-18
Table 6-5: Infiltration Trench Design Criteria... 6-27
Table 6-6: Bioretention Design Criteria 6-36
Table 6-7: Suitability Assessment Related Considerations for Infiltration Facility Safety Factors 6-38
Table 6-8: Design Related Considerations for Infiltration Facility Safety Factors 6-39
Table 6-9: Infiltration Facility Safety Factor Determination Worksheet.............................. 6-40
Table 6-10: Infiltration BMP Design Criteria 6-52
Table 6-11: Permeable Pavements Design Criteria 6-60
Table 6-12: Suitability Assessment Related Considerations for Infiltration Facility Safety Factors 6-63
Table 6-13: Design Related Considerations for Infiltration Facility Safety Factors 6-65
Table 6-14: Infiltration Facility Safety Factor Determination Worksheet............................ 6-66
Table 6-15: Proprietary Infiltration Manufacturer Websites.. 6-75
Table 6-16: Bioretention Design Criteria 6-81
Table 6-17: Green Roof Design Criteria 6-104
Table 6-18: Bioretention with an Underdrain Design Criteria 6-118
Table 6-19: Planter Box Design Criteria 6-131
Table 6-20: Vegetated Swale Filter Design Criteria 6-141
Table 6-21: Vegetated Filter Strip Design Criteria... 6-156
Table 6-22: Proprietary Biotreatment Device Manufacturer Websites 6-165
Table 6-23: Dry Extended Detention Basin Design Criteria... 6-171
Table 6-24: Wet Detention Basin Design Criteria... 6-191

Table 6-25: Constructed Wetland Design Criteria 6-209
Table 6-26: Sand Filter Design Criteria 6-226
Table 6-27: Proprietary Cartridge Media Filter Manufacturer Websites 6-238
Table 6-28: Proprietary Hydrodynamic Device Manufacturer Websites 6-242
Table 6-29: Proprietary Catch Basin Insert Manufacturer Websites 6-246

LIST OF FIGURES

Figure 2-1: Stormwater Management Control Measures Design Decision Flowchart 2-2
Figure 2-2: Apply BMPs to Reduce EIA to ≤5% Process Flow Chart 2-10
Figure 2-3: Alternative Stormwater Management Control Measures Compliance Decision Flow Chart 2-19
Figure 5-1: Storm Drain Message Location 5-4
Figure 6-1: Differences between On-line, Off-line, and In-stream Control Measures 6-6
Figure 6-2: Infiltration Basin Design Schematic 6-9
Figure 6-3: Infiltration Trench Design Schematic 6-25
Figure 6-4: Bioretention Design Schematic 6-34
Figure 6-5: Drywell Design Schematic 6-50
Figure 6-6: Permeable Pavement Design Schematic 6-58
Figure 6-7: Proprietary Infiltration BMP Design Schematic 6-73
Figure 6-8: Bioinfiltration Design Schematic 6-79
Figure 6-9: Green Roofs 6-103
Figure 6-10: Bioretention with Underdrain Design Schematic 6-117
Figure 6-11: Planter Box Design Schematic 6-130
Figure 6-12: Vegetated Swale Design Schematic 6-140
Figure 6-13: Vegetated Filter Strip Design Schematic 6-155
Figure 6-14: Biotreatment Device Design Schematic 6-164
Figure 6-15: Dry Extended Detention Basin Schematic 6-167
Figure 6-16: Perforated Riser Outlet 6-168
Figure 6-17: Multiple Orifice Outlet 6-169
Figure 6-18: Spillway 6-170
Figure 6-19: Wet Detention Basin Schematic 6-188
Figure 6-20: Riser Outlet 6-189
Figure 6-21: Inverted Pipe Outlet 6-190
Figure 6-22: Constructed Wetland Schematic 6-208
Figure 6-23: Sand Filter Design Schematic 6-225
Figure 6-24: Cartridge Media Filter Design Schematic 6-237
Figure 6-25: Hydrodynamic Separation Device Design Schematic 6-241
Figure 6-26: Catch Basin Insert Design Schematic 6-245

APPENDICES

Appendix A	Glossary of Terms
Appendix B	Maps: Watersheds Delineation, Existing Urban Areas, and Environmentally Sensitive Areas
Appendix C	Site Soil Type and Infiltration Testing
Appendix D	BMP Performance Guidance
Appendix E	BMP Sizing Worksheets
Appendix F	Flow Splitter Design
Appendix G	Design Criteria Checklists for Stormwater Runoff BMPs
Appendix H	Stormwater Control Measure Access and Maintenance Agreements
Appendix I	Stormwater Control Measure Maintenance Plan Guidelines and Checklists

1 INTRODUCTION

This *Technical Guidance Manual for Stormwater Quality Measures* (2011 TGM) provides guidance for the implementation of stormwater management control measures in new development and redevelopment projects in the County of Ventura and the incorporated cities therein. These guidelines are intended to improve water quality and mitigate potential water quality impacts. These guidelines have been developed to meet the Planning and Land Development requirements contained in Part 4, Section E of the Los Angeles Regional Water Quality Control Board's (Regional Board) municipal separate storm sewer system (MS4) permit (Order R4-2010-0108) for new development and redevelopment projects.

The Planning and Land Development requirements are not implemented at the discretion of the local permitting agency; they are requirements in Order R4-2010-0108 that must be complied with. The 2011 TGM does not attempt to expand or circumvent these requirements, but rather it provides guidance on how to meet them.

When used in this Manual, the verb "shall" indicates a statement of required, mandatory, or specifically prohibited practice. Statements that are not mandatory, but are recommended practice in typical situations, with allowable deviations if engineering judgment or scientific study indicates them appropriate, are typically stated with the verb "should." In both cases specific options may be provided that are allowable modifications.

1.1 Goals

The 2011 TGM has been prepared by the Ventura Countywide Stormwater Quality Management Program to accomplish the following goals:

- Ensure that new development and redevelopment projects reduce urban runoff pollution to the "maximum extent practicable" (MEP);
- Ensure that the implementation of measures in the 2011 TGM are consistent with Regional Water Quality Control Board Order R4-2010-0108 and other state requirements;
- Provide guidance to developers, design engineers, agency engineers, and planners on the selection and implementation of appropriate stormwater management control measures; and
- Provide maintenance procedures to ensure that the selected stormwater management control measures will be properly maintained to provide effective, long-term pollution control.

1.2 Regulatory Background

In 1972, the Federal Water Pollution Control Act [later referred to as the Clean Water Act (CWA)] was amended to require National Pollutant Discharge Elimination System (NPDES) permits for the discharge of pollutants to waters of the United States from any point source. In 1987, the CWA was amended to require the United States Environmental Protection Agency (USEPA) to establish regulations permitting municipal and industrial stormwater discharges under the NPDES permit program. The USEPA published final regulations regarding stormwater discharges on November 16, 1990. The regulations require that MS4 discharges to surface waters be regulated by a NPDES permit.

The Ventura County Watershed Protection District, County of Ventura, and the cities of Camarillo, Fillmore, Moorpark, Ojai, Oxnard, Port Hueneme, San Buenaventura, Santa Paula, Simi Valley, and Thousand Oaks have joined together to form the Ventura Countywide Stormwater Quality Management Program (Program)and are named as co-permittees under a revised countywide municipal NPDES permit for stormwater discharges issued by the Regional Water Quality Control Board in 2010 (Order R4-2010-0108).

Prior to the issuance of Order R4-2010-0108, stormwater discharges from the Ventura County MS4 were covered under the countywide waste discharge requirements contained in three previous MS4 NPDES Permits (Order 09-0057, Order 00-108, and Order No. 94-082).

Under Order R4-2010-0108, the co-permittees are required to administer, implement, and enforce a Stormwater Quality Management Program (Program) to reduce pollutants in urban runoff to the MEP. The Program emphasizes all aspects of pollution control including, but not limited to, public awareness and participation, source control, regulatory restrictions, water quality monitoring, and treatment control.

For the Program to be successful, it is critical to control urban runoff pollution from new development and redevelopment projects during and after construction. Therefore, the co-permittees implemented the Planning and Land Development Program, one element within the Program, to specifically control post-construction urban runoff pollutants from new development and redevelopment projects. The goal of the Planning and Land Development Program is to minimize runoff pollution typically caused by land development and protect the beneficial uses of receiving waters by limiting effective impervious area (EIA) to no more than 5% of the project area and retaining stormwater on site. This goal can be achieved by employing a sensible combination of Site Design Principles and Techniques, Source Control Measures, Retention Best Management Practices (BMPs), Biofiltration BMPs, and Treatment Control Measures to the level required in Order R4-2010-0108.

"Site Design Principles and Techniques," "Source Control Measures," "Retention

BMPs," "Biofiltration BMPs," and "Treatment Control Measures," as used in the 2011 TGM refer to BMPs and features incorporated into the design of a new development or redevelopment project, which prevent and/or reduce pollutants in stormwater runoff from the project. These measures are described below:

1) ***Site Design Principles and Techniques*** are a stormwater management strategy that emphasizes conservation and use of existing site features to reduce the amount of runoff and pollutant loading that is generated from a project site.

2) ***Source Control Measures*** limit the exposure of materials and activities so that potential sources of pollutants are prevented from making contact with stormwater runoff.

3) ***Retention BMPs*** are stormwater BMPs that are designed to retain water onsite, and achieve a greater reduction in surface runoff from a project site than traditional stormwater Treatment Control Measures. The term "Retention BMPs" encompasses infiltration, rainwater harvesting[1], and evapotranspiration BMPs. Retention BMPs are preferred and shall be selected over biofiltration BMPs and Treatment Control Measures where technically feasible to do so.

4) ***Biofiltration BMPs*** are vegetated stormwater BMPs that remove pollutants by filtering stormwater through vegetation and soils.

5) ***Treatment Control Measures*** are engineered BMPs that provide a reduction of pollutant loads and concentrations in stormwater runoff.

Applicable projects (Section 1.4) must reduce Effective Impervious Area (EIA) to less than or equal to five percent (≤5%) of the total project area, unless infeasible. Impervious surfaces are rendered "ineffective" if the design storm volume is fully retained onsite using Retention BMPs. Biofiltration BMPs may be used to achieve the 5% EIA standard if Retention BMPs are technically infeasible (see Section 3.2).

The 2011 TGM contains guidance for the design and implementation of all of these types of stormwater management control measures for new development and redevelopment projects. In addition to the requirements of Order R4-2010-0108, owners and developers of some of the sites in the County may also be subject to the State of California's general permit for stormwater discharge from industrial activities (Industrial General Permit) and general permit for stormwater discharge from construction activities (Construction General Permit). The stormwater management control measures provided in the 2011 TGM may also assist the owner or developer in meeting the requirements of the State's construction and industrial permits. The stormwater management staffs of the governing co-permittee agencies are available to provide assistance regarding all of the State stormwater permit

[1] Rainwater harvesting is a BMP that stores and uses rainwater or stormwater runoff. This is consistent with the use of the term "reuse" contained in Order R4-2010-0108.

requirements.

1.3 Impacts of Land Development

The Cities and County of Ventura have separate stormwater and sanitary sewer conveyance systems. Land development typically creates an increase in impervious surfaces, which increases the amount of runoff and pollutants entering stormwater conveyance systems. Pollutants that enter the conveyance system in stormwater are typically transported directly to receiving waters (i.e. local channels, rivers, and the ocean), and are not treated in a wastewater treatment plant. Pollutants in untreated stormwater runoff from impervious surfaces that drains to streets and enters storm drains directly contribute to water pollution.

Typically, as stormwater runs over impervious surfaces (e.g., rooftops, roadways, and parking lots), it:

- Does not infiltrate or evapotranspire, which increases runoff volumes, velocities, and flow rates;
- Moves more quickly, which increases runoff velocities; and
- Entrains (i.e., accumulates) pollution and sediment, which increases nutrients, bacteria, and other pollutant concentrations in receiving waters (i.e., local channels, rivers, and the ocean).

The impacts of these alterations due to development may include:

- Increased concentrations of nutrients, toxic pollutants, and bacteria in surface receiving waters, including adjacent land and habitat (e.g., beaches) creeks, estuaries, and storm drain outlets.
- Increased flooding due to higher peak flow rates and runoff volumes produced by a storm.
- Decreased wet season groundwater recharge due to a decreased infiltration area.
- Increased dry season groundwater recharge due to outdoor irrigation with potable or reclaimed water.
- Introduction of baseflows in ephemeral streams due to surface discharge of dry weather urban runoff.
- Increased stream and channel bank instability and erosion due to increased runoff volumes, flow durations, and higher stream velocities ("hydromodification impacts"); and

- Increased stream temperature due to loss of riparian vegetation as well as runoff warmed by impervious surfaces, which decreases dissolved oxygen levels and makes streams inhospitable to some aquatic life requiring cooler temperatures for survival.

1.4 Stormwater Management Principles

Stormwater management principles such as Integrated Water Resource Management (IWRM) and Low Impact Development (LID) can be used to help mitigate the impacts of development. These principles are described below.

The emergence of LID falls under the umbrella of the over-arching concept of IWRM. IWRM is a process which promotes the coordinated development and management of water, land, and related resources. IWRM links traditional development topics such as land use, water supply, wastewater treatment/reclamation, flood control/drainage, water quality, and hydromodification management into a cohesive hydrologic system that recognizes their interdependencies and minimizes their potentially negative effects on the environment. An example of IWRM includes recharging groundwater with reclaimed wastewater to support the water supply. Another example is combining stormwater treatment, hydromodification control, and flood control in a single regional infiltration basin that recharges groundwater, incorporates recreation, and provides habitat. Another example is using Smart Growth principles to help reduce the environmental footprint while still accommodating growth.

Generally, the 2011 TGM advises to first design for the largest hydrologic controls (such as matching post development 100-year flows with pre-project 100-year flows for flood mitigation requirements), according to the appropriate City or County drainage requirements (not included in the 2011 TGM). Secondly, the 2011 TGM advises to check if flood mitigation will reduce or satisfy the stormwater management requirements (as set forth in the 2011 TGM). If it does not, then add more controls as necessary. Flood mitigation may provide the necessary sediment and pollution control, thereby reducing maintenance requirements for the stormwater management BMPs. A sequence of hydrologic controls should be considered, such as site design, flood drainage mitigation, and Retention BMPs. Biofiltration BMPs and Treatment Control Measures can be considered where the use of Retention BMPs is technically infeasible. Each of these controls will have an influence on stormwater runoff from the new development or redevelopment project.

Similar to Source Control Measures, which prevent pollutant sources from contacting stormwater runoff, Retention BMPs use techniques to infiltrate, store, use, and evaporate runoff onsite to mimic pre-development hydrology, to the extent feasible. The goal of LID is to increase groundwater recharge, enhance water quality, and prevent degradation of downstream natural drainage channels. This goal may be accomplished with creative site planning and with incorporation of localized, naturally functioning BMPs into the project. Implementation of Retention BMPs will reduce the size of additional Hydromodification Control Measures that may be required for a new development or redevelopment project, and, in many circumstances, may be used to satisfy all stormwater management requirements.

1.5 Applicability

The following projects and associated triggers, contained in subpart 4.E.II of Order R4-2010-0108, are subject to the requirements and standards laid out in the 2011 TGM.

Note that some of the project triggers are based on *total altered surface area* and others on *impervious surface area*, which is an intentional requirement in the MS4 Permit.

New Development Projects

Development projects subject to conditioning and approval for the design and implementation of post-construction stormwater management control measures, prior to completion of the project(s), are:

1) All development projects equal to 1 acre or greater of disturbed area that adds more than 10,000 square feet of impervious surface area.

2) Industrial parks with 10,000 square feet or more of total altered surface area.

3) Commercial strip malls with 10,000 square feet or more of impervious surface area.

4) Retail gasoline outlets with 5,000 square feet or more of total altered surface area.

5) Restaurants (Standard Industrial Classification (SIC) of 5812) with 5,000 square feet or more of total altered surface area.

6) Parking lots with 5,000 square feet or more of impervious surface area, or with 25 or more parking spaces.

7) Streets, roads, highways, and freeway construction of 10,000 square feet or more of impervious surface area (see Section 2 for specific requirements).

8) Automotive service facilities (Standard Industrial Classification (SIC) of 5013, 5014, 5511, 5541, 7532-7534 and 7536-7539) of 5,000 square feet or more of total altered surface area.

9) Projects located in or directly adjacent to, or discharging directly to an Environmentally Sensitive Area (ESA), where the development will:

    a. Discharge stormwater runoff that is likely to impact a sensitive biological species or habitat; and

b. Create 2,500 square feet or more of impervious surface area.

10) Single-family hillside homes (see Section 2 for specific requirements).

Redevelopment Projects

Redevelopment projects subject to conditioning and approval for the design and implementation of post-construction stormwater management control measures, prior to completion of the project(s), are redevelopment projects in categories 1 through 10 above that meet the threshold identified below:

- Land-disturbing activity that results in the creation or addition or replacement of 5,000 square feet or more of impervious surface area on an already developed site.

Additionally:

1) Projects where redevelopment results in an alteration to more than fifty percent of impervious surfaces of a previously existing development, and the existing development was not subject to the post development stormwater quality control requirements of Board Order 00-108, shall mitigate the entire redevelopment project area.

2) Projects where redevelopment results in an alteration to more than fifty percent of impervious surfaces of a previously existing development, and the existing development was subject to the post development stormwater quality control requirements of Board Order 00-108, must mitigate only the altered portion of the redevelopment project area and not the entire project area.

3) Projects where redevelopment results in an alteration of less than fifty percent of impervious surfaces of a previously existing development must mitigate only the altered portion of the redevelopment project area and not the entire project area.

Land-disturbing activity that results in the creation or addition or replacement of less than 5,000 square feet of impervious surface area on an already developed site, or that results in a decrease in impervious area which was subject to the post-development stormwater quality control requirements of Board Order 00-108, is not subject to mitigation unless so directed by the local permitting agency.

Redevelopment does not include routine maintenance activities that are conducted to maintain the original line and grade, hydraulic capacity, or original purpose of the facility or emergency redevelopment activity required to protect public health and safety. Impervious surface replacement, such as the reconstruction of parking lots and roadways, that does not disturb additional area and maintains the original grade and alignment, is considered a routine maintenance activity. Agencies' flood control, drainage, and wet utilities projects that maintain original line and grade or hydraulic capacity are considered routine maintenance. Redevelopment also does not include the repaving of existing roads to maintain original line and grade.

Existing single-family dwelling and accessory structure projects are exempt from the redevelopment requirements unless the project creates, adds, or replaces 10,000 square feet of impervious surface area.

Effective Date

The new development and redevelopment requirements contained in Part 4, Section E of Board Order R4-2010-0108 (the "Order") shall become effective 90 calendar days after the Regional Water Quality Control Board Executive Officer approves the 2011 TGM (the "Effective Date"). After the Effective Date, all applicable projects, except those identified below, must comply with the new development and redevelopment requirements contained in Part 4, Section E of the Order.

The new development and redevelopment requirements contained in Part 4, Section E of the Order shall not apply to the projects described in paragraphs 1 through 5 below. Projects meeting the criteria listed in paragraphs 1 through 5 below shall instead continue to comply with the performance criteria set forth in the 2002 Technical Guidance Manual for Stormwater Quality Control Measures under Board Order 00-108:

1) Projects or phases of projects where the project's applications have been "deemed complete for processing" (or words of equivalent meaning), including projects with ministerial approval, by the applicable local permitting agency in accordance with the local permitting agency's applicable rules prior to the Effective Date; or

2) Projects that are the subject of an approved Development Agreement and/or an adopted Specific Plan; or an application for a Development Agreement and/or Specific Plan where the application for the Development Agreement and/or Specific Plan has been "deemed complete for processing" (or words of equivalent meaning), by the applicable local permitting agency in accordance with the local permitting agency's applicable rules, and thereafter during the term of such Development Agreement and/or Specific Plan unless earlier cancelled or terminated; or

3) All private projects in which, prior to the Effective Date, the private party has completed public improvements; commenced design, obtained financing, and/or participated in the financing of the public improvements; or which requires the private party to reimburse the local agency for public improvements upon the development of such private project; or

4) Local agency projects for which the governing body or their designee has approved initiation of the project design prior to the Effective Date; or

5) A Tentative Map or Vesting Tentative Map deemed complete or approved by the local permitting agency prior to the Effective Date, and subsequently a Revised Map is submitted, the project would be exempt from the 2011 TGM provisions if the revisions substantially conform to original map design, consistent with

Subdivision Map Act requirements. Changes must also comply with local and state law.

The intent of these guidelines is to ensure that projects for which the applications have been deemed "complete" or the applicants have worked with local permitting agency staff to develop a final, or substantially final, drainage concept and site layout that includes water quality treatment based upon the performance criteria set forth in the 2002 Technical Guidance Manual for Stormwater Quality Control Measures prior to the Effective Date, are not required to redesign their proposed projects for purposes of complying with the new development and redevelopment requirements contained in Part 4, Section E of Board Order R4-2010-0108.

In addition, any project, phase of a project, or individual lot within a larger previously-approved project, where the application for such project has been "deemed complete for processing" (or words of equivalent meaning) that does not have a final or substantially final drainage concept as determined by the local permitting agency or a site layout that includes water quality treatment must comply with the performance standards set forth in the 2011 TGM.

1.6 Organization of the 2011 TGM

The 2011 TGM is divided into seven sections and nine appendices:

Section 1	Introduction
Section 2	Stormwater Management Standards
Section 3	Site Assessment and BMP Selection
Section 4	Site Design Principles & Techniques
Section 5	Source Control Measures
Section 6	Retention BMPs, Biofiltration BMPs, and Treatment Control Measure Design
Section 7	Operation and Maintenance Planning
Appendix A	Glossary of Terms
Appendix B	Maps: Watersheds Delineation, Existing Urban Areas, Environmentally Sensitive Areas, and 85th Percentile Rainfall Depth
Appendix C	Site Soil Type and Infiltration Testing

Appendix D	BMP Performance Guidance
Appendix E	BMP Sizing Worksheets
Appendix F	Flow Splitter Design
Appendix G	Design Criteria Checklists for Stormwater Runoff BMPs
Appendix H	Stormwater Control Measure Access and Maintenance Agreements
Appendix I	Stormwater Control Measure Maintenance Plan Guidelines and Checklists

2 STORMWATER MANAGEMENT STANDARDS

2.1 Introduction

This section outlines the design process to comply with stormwater control requirements. A flowchart is presented in Figure 2-1 to illustrate a step-by-step process for incorporating these stormwater management control measures.

The selection of appropriate stormwater management control measures should be a collaborative effort between the project proponent and the local permitting agency staff. It is recommended that discussions between project planners, engineers, and local permitting agency staff regarding selection of stormwater management control measures occur very early in the design process.

2.2 Step 1: Determine Project Applicability

New development and redevelopment projects meeting the applicability criteria contained in Section 4.E.II of Order R4-2010-0108 [presented in Section 1.5 of the 2011 TGM] must include control measures specified in the 2011 TGM. These projects should be designed to meet the performance criteria described in the steps below.

Separate requirements exist for three types of projects:

- Projects located within a Redevelopment Project Area Master Plan (RPAMP);
- Single Family Hillside Homes; and
- Roadway Projects.

The requirements for these three project types are described in further detail in the substeps below. Projects that are not applicable are still subject to stormwater agency review, especially for flood drainage requirements. Stormwater management control measures may be required by the governing agency for inapplicable projects, depending on the potential discharge of pollutants in stormwater runoff, impairments in receiving water, or other special conditions that would require increased protection.

Step 1a: Determine RPAMP Eligibility

If a project is located within the boundary of a Redevelopment Project Area Master Plan (RPAMP), the stormwater management requirements in the RPAMP take precedence over the control measures and performance criteria specified in this 2011 TGM. A stormwater agency may apply to the Regional Water Quality Control Board for approval of a RPAMP in consideration of exceptional site constraints that inhibit site-by-site or project-by-project implementation of post-construction requirements.

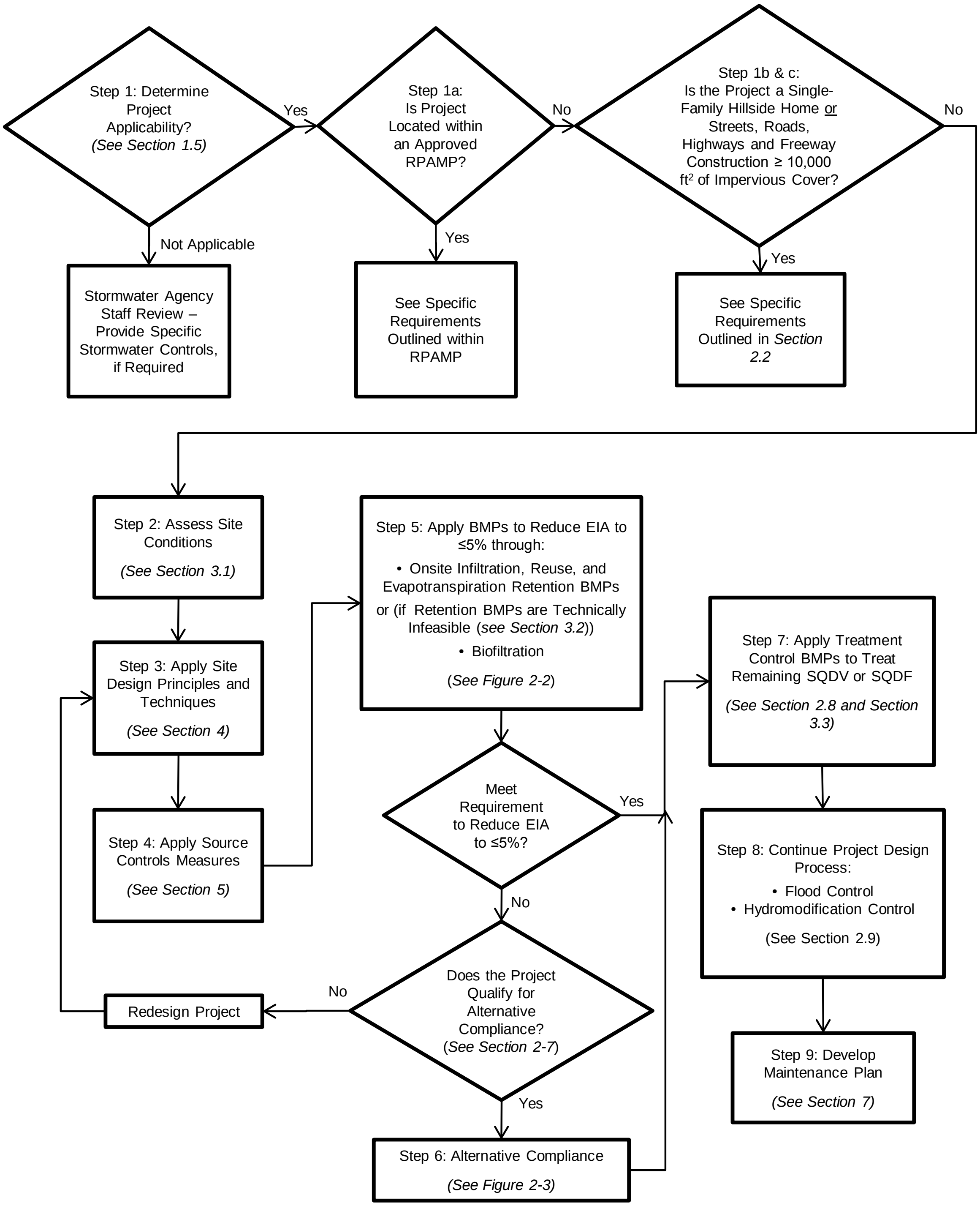


Figure 2-1: Stormwater Management Control Measures Design Decision Flowchart

Step 1b: Single-Family Hillside Homes

Single-family hillside home projects have specific requirements separate from other new development and redevelopment project categories. These requirements only apply to single-family hillside homes that disturb less than 1 acre and that add less than 10,000 square feet of impervious surface area. If the project is equal to 1 acre or greater of disturbed area that adds more than 10,000 square feet of impervious surface area, then project must comply with Steps 2 through 9.

According to Order R4-2010-0108, a hillside is defined as:

> *"Property located in an area with known erosive soil conditions, where the development will result in grading on any slope that is 20% or greater or an area designated by the Municipality under a General Plan or ordinance as a 'hillside area.'"*

The measures presented in this substep comprise the performance standard for single-family hillside home new development and redevelopment projects and apply to the entire lot (additional information on these measures may be found in Section 4 and Section 5).

Conserve Natural Areas

Each project site possesses unique topographic, hydrologic and vegetative features, some of which are more suitable for development than others. Locating development on the least sensitive portion of a site and conserving naturally vegetated areas can minimize environmental impacts in general and stormwater runoff impacts in particular.

The following measures are required and should be included in the lot layout, consistent with applicable General Plan and Local Area Plan policies and if appropriate and feasible with the given site conditions:

1) Concentrate or cluster improvements on the least-sensitive portions of the lot and leave the remaining land in a natural undisturbed state; at a minimum, sensitive portions of the lot should include areas covered under Clean Water Act Section 404 such as riparian areas and wetlands;

2) Limit clearing and grading of native vegetation on the lot to the minimum area needed to build the home, allow access, and provide fire protection; and

3) Maximize trees and other vegetation at the site by planting additional vegetation, clustering tree areas, and promoting the use of native and/or drought-tolerant plants.

Protect Slopes and Channels

Erosion of slopes and channels can be a major source of sediment and associated pollutants such as nutrients, if not properly protected and stabilized.

Slope Protection

Slope protection practices must conform to local permitting agency erosion and sediment control standards and design requirements. The post-construction design criteria described below are intended to enhance and be consistent with these local standards.

1) Slopes must be protected from erosion by safely conveying runoff from the tops of slopes.

2) Slopes must be vegetated by first considering the use of native or drought-tolerant species.

Channel Protection

The following measures should be implemented to provide erosion protection to unlined receiving streams on the lot. Activities and structures must conform to applicable permitting requirements, standards, and specifications of agencies with jurisdiction (i.e., U.S. Army Corps of Engineers, California Department of Fish and Game, or Regional Water Quality Control Board).

1) Use natural drainage systems to the maximum extent practicable, but minimize runoff discharge to the maximum extent practicable.

2) Stabilize permanent channel crossings.

3) Install energy dissipaters, such as rock riprap, at the outlets of storm drains, culverts, conduits or channels that discharge into unlined channels.

Provide Storm Drain System Stenciling and Signage

Storm drain message markers or placards are required at all storm drain inlets within the project boundary. The signs should be placed in clear sight facing anyone approaching the inlet from either side. All storm drain inlet locations must be identified on the development site map.

Some local agencies within the County have approved storm drain message placards for use. Consult local permitting agency stormwater staff to determine specific requirements for placard types and installation methods.

Divert Roof Runoff and Surface Flows to Vegetated Area(s) or Collection System(s), Unless the Diversion Would Result in Slope Instability

Diverted Roof Runoff
City of Santa Barbara

Disconnecting downspouts divert water from roof gutters to (1) vegetated pervious areas of the site in order to allow for infiltration, storage, evapotranspiration (i.e., evaporation and uptake of water by plants), and treatment, or (2) a rainwater collection system (e.g., a rain barrel or a cistern). Disconnected downspouts differ from conventional downspout systems that provide a direct connection of roof runoff to stormwater conveyance systems (storm drains), which quickly collect and convey stormwater away from the site. "Flow spreading" is a technique used to spread runoff from rooftops, sidewalks, patios, and driveways out over a vegetated pervious area, rather than concentrating and conveying the runoff directly to a stormwater conveyance system.

Dispersion methods include splash blocks, gravel-filled trenches, or other methods which serve to spread runoff over vegetated pervious areas. Sheet flow dispersion is the simplest method and can be used for any impervious or pervious surface that is graded so as to avoid concentrating flows. Because flows are already dispersed as they leave the surface, they only need to traverse through a narrow band of adjacent vegetation for the runoff to be effectively attenuated and treated.

The following requirements apply to runoff diversion:

- Vegetated flowpaths for the diverted flows should be at least 25 feet in length, measured from the diversion location to the downstream property line, structure, steep slope, stream, wetland, or impervious surface. The vegetated flowpath must be covered with well-established lawn or pasture, landscaping with well-established groundcover, or native vegetation with natural groundcover. The groundcover should be dense enough to help disperse and infiltrate flows and to prevent erosion.

- If the vegetated flowpath (measured as defined above) is less than 25 feet, a perforated stub-out connection may be used in lieu of downspout dispersion. A perforated stub-out connection is a length of perforated pipe within a gravel-filled trench that is placed between roof downspouts and a stub-out to the local drainage system. A perforated stub-out may also be used where implementation of downspout dispersion might cause erosion or flooding problems, either onsite or on adjacent lots. This provision might be appropriate, for example, for lots where dispersed flows might pose a potential hazard for lower lying lots or adjacent offsite lots. Location of the connection should be selected to allow a maximum amount of runoff to infiltrate into the ground (ideally a dry location on the site that is relatively well drained). To facilitate maintenance, the perforated pipe portion of the system should not be located under impervious or heavily compacted (e.g., driveways and parking areas) surfaces. The use of a perforated stub-out in lieu of downspout dispersion may be determined by the Local permitting agency.

- In general, if the ground is sloped away from the foundation and there is adequate vegetation and area for effective dispersion, splash blocks will adequately disperse stormwater runoff. If the ground is fairly level, if the structure includes a basement, or if foundation drains are proposed, splash blocks with downspout extensions may be a better choice because the discharge point is moved away from the foundation. Downspout extensions may include piping to a splash block/discharge point a considerable distance from the downspout, as long as the runoff can travel through a well-vegetated area as described above.

- No erosion or flooding of downstream properties may result.

- Runoff discharged towards steep slopes or landslide hazard areas, including perforated stub-out connections, must be evaluated by a geotechnical engineer or qualified geologist. The discharge point may not be placed on or above slopes greater than 20% or above erosion hazard areas without evaluation by a geotechnical engineer or qualified geologist and jurisdiction approval.

- For sites with septic systems, the discharge point must be down gradient of the drainfield primary and reserve areas. This requirement can be waived by the jurisdiction's permit review staff if site topography clearly prohibits flows from intersecting with the drainfield.

Step 1c: Roadway Projects

Roadway projects have specific requirements separate from other new development and redevelopment project categories. The measures presented in this substep comprise the performance standard for street, roadway, highway, and freeway projects. Section 4.E.II of Order R4-2010-0108 requires street, roadway, highway, and freeway projects that construct 10,000 square feet or more of impervious surface area, to incorporate USEPA guidance regarding *Managing Wet Weather with Green Infrastructure: Green Streets* to the maximum extent practicable.

The following requirements apply to the impervious area within the right-of-way associated with public streets, roads, highways, and freeways projects and the streets that are part of a larger private project. These requirements do not apply to routine maintenance activities that are conducted to maintain original line and grade, hydraulic capacity, original purpose of facility, or emergency redevelopment activity required to protect public health and safety. Impervious surface replacement, such as the reconstruction of parking lots and roadways, which does not disturb additional area and maintains the original grade and alignment, is considered a routine maintenance activity. Agencies' flood control, drainage, and wet utilities projects that maintain original line and grade or hydraulic capacity are considered routine maintenance. Also, the requirements do not apply to the repaving of existing roads to maintain original line and grade.

Minimum requirements for the impervious area within the right-of-way associated with streets, roads, highways, and freeways are as follows:

1) Provide Retention BMPs or Biofiltration BMPs sized to capture and treat the Stormwater Quality Design Volume (SQDV) or the Stormwater Quality design Flow (SQDF) (see Step 7 for guidance on calculating the SQDV and SQDF).

   Additional Treatment Control Measures may be integrated into roadway projects if they are used in a treatment train approach with Retention BMPs or Biofiltration BMPs to address the pollutants of concern (see Section 3.3).

2) Projects should apply the following measures to the maximum extent practicable and as specified in the local permitting agency's codes:

   - Minimize street width to the appropriate minimum width for maintaining traffic flow and public safety;
   - Use porous pavement or pavers for low traffic roadways, on-street parking, shoulders or sidewalks; and
   - Add tree canopy by planting or preserving trees and shrubs.

2.3 Step 2: Assess Site Conditions

The next step is to collect site information that is critical for the selection and implementation of Retention BMPs, Biofiltration BMPs, and Treatment Control Measures. The following information should be documented: topography, soil type and geology, groundwater, geotechnical considerations, offsite drainage, existing utilities, and Environmentally Sensitive Areas. In addition, soil and infiltration testing should be conducted. Detailed guidance on assessing site conditions can be found in Section 3.1.

2.4 Step 3: Apply Site Design Principles and Techniques

The third step is to apply Site Design Principles & Techniques (see Section 4). The implementation of LID requires an integrated approach to site design and stormwater management. Traditional approaches to stormwater management planning within the site planning process are not likely to achieve the LID performance standard of the MS4 Permit. The use of the site planning techniques presented in Section 4 (Site Design Principles & Techniques) will help generate a more hydrologically functional site, maximize the effectiveness of Retention BMPs, and integrate stormwater management throughout the site.

The following criteria should be considered during the early site planning stages:

- Retention BMPs should be considered as early as possible in the site planning process. Hydrology should be a key principle that is integrated into the initial site assessment planning phases. Where flexibility exists, conceptual drainage plans should attempt to route water to areas suitable for Retention BMPs.

- A multidisciplinary approach at the initial phases of the project is recommended and should include planners, engineers, landscape architects, and architects.

- Individual Retention BMPs should be distributed throughout the project site as feasible and may influence the configuration of roads, buildings and other infrastructure.

- The project must demonstrate disconnection of impervious surface such that the 5% EIA requirement is achieved. If fully meeting the 5% EIA requirement using Retention BMPs is not technically feasible, the project must still utilize Retention BMPs to the maximum extent practicable.

- Flood and hydromodification control should be considered early in the design stages. Even sites with Retention BMPs will still have runoff that occurs during large storm events, but Retention facilities can have flood and hydromodification control benefits. It may be possible to simultaneously address flood and hydromodification control requirements through an integrated water resources management approach.

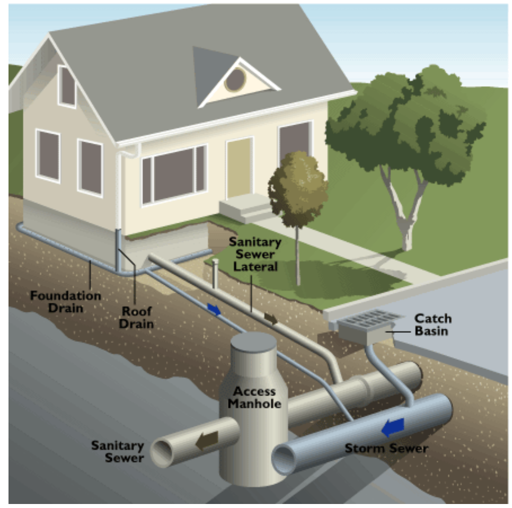
Perhaps the most important aspect of site planning is allowing sufficient space for Retention BMPs in areas that can physically accept runoff. A simple rule of thumb is to allow 3 to 10 percent of the tributary impervious area (depending on how well the soils drain and then allow for more area with less infiltrative soils) for infiltration BMPs and 3 to 5 percent for biofiltration in preliminary design to achieve the 5% Effective Impermeable Area (EIA) standard.

2.5 Step 4: Apply Source Control Measures

All applicable projects must implement applicable Source Control Measures. Source Control Measures are operational practices that reduce potential pollutants at the source. They typically do not require maintenance or significant construction. Guidance on Source Control Measures can be found in Section 5.

2.6 Step 5: Apply BMPs to Reduce EIA to ≤5%

According to Order R4-2010-0108, Applicable projects must reduce Effective Impervious Area (EIA) to less than or equal to five percent (≤5%) of the total project area, unless infeasible. Impervious surfaces are rendered "ineffective" if the design storm volume is fully retained onsite using either infiltration, rainwater harvesting, and/or evapotranspiration Retention BMPs. Biofiltration BMPs may be used to achieve the 5% EIA standard if Retention BMPs are technically infeasible (see Section 3.2). This section and Figure 2-2 describe the process for reducing EIA to ≤5%. Refer to Section 2.7 if Retention BMPs and/or Biofiltration BMPs cannot feasibly be used to meet the 5% EIA standard (see Section 3.2).



Effective Impervious Area
Victoria, BC Capital Regional District

Step 5a: Calculate Allowable EIA

EIA is defined as impervious area that is hydrologically connected via sheet flow over a hardened conveyance or impervious surface without any intervening medium to mitigate flow volume. Connected impervious areas efficiently transport runoff without allowing infiltration. Often in urban areas, runoff from connected impervious surfaces is immediately directed into a stormwater conveyance system where it is further connected and efficiently transported to an outfall (stormwater conveyance system outlet). For example, in this illustration, the rooftop is directly connected via a roof drain and underground solid drain pipe to the storm drain in the street (Note that the sanitary sewer is separate from the storm sewer). The roadway drains to the storm drain through the catch basin. The roof area and roadway area would be considered EIA.

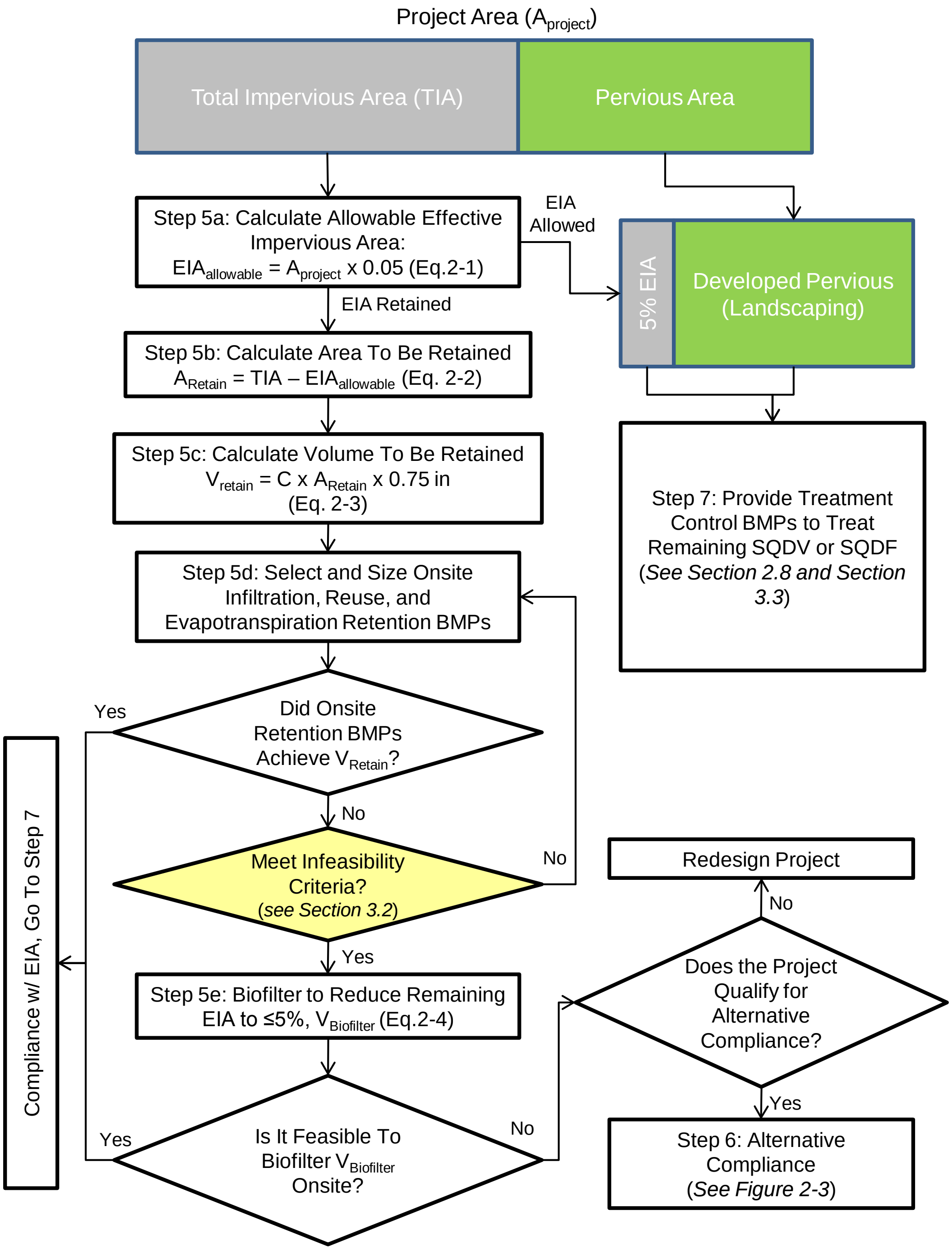


Figure 2-2: Apply BMPs to Reduce EIA to ≤5% Process Flow Chart

"Impervious surface" is a man-made hard surface area which causes water to run off the surface in greater quantities or at an increased rate of flow from the flow present under natural conditions prior to development. Common impervious surfaces include, but are not limited to, rooftops, walkways, patios, driveways, parking lots or storage areas, concrete or asphalt paving, compacted gravel roads, packed earthen materials, and oiled, macadam or other surfaces which similarly impede the natural infiltration of stormwater. Open, uncovered retention/detention facilities and exposed bedrock shall not be considered as impervious surfaces for purposes of determining EIA retention volume.

The allowable EIA for a project site should be calculated as follows:

$$EIA_{allowable} = (A_{project})*(\%_{allowable}) \qquad \text{(Equation 2-1)}$$

Where:

$EIA_{allowable}$ = the maximum impervious area from which runoff can be treated and discharged offsite [and not retained onsite] (acres)

$A_{project}$ = the total project area (acres).

"Total project area" (or "gross project area") for new development and redevelopment projects is defined as the disturbed, developed, and undisturbed portions within the project's property (or properties) boundary, at the project scale submitted for first approval. Areas proposed to be permanently dedicated for open space purposes as part of the project are explicitly included in the "total project area." Areas of land precluded from development through a restrictive covenant, conservation easement, or other recorded document for the permanent preservation of open space prior to project submittal shall not be included in the "total project area."

$\%_{allowable}$ = 5 percent

Step 5b: Calculate Impervious Area to be Retained

The impervious area from which runoff must be retained onsite is the total impervious area minus the $EIA_{allowable}$, which should be calculated as follows:

$$A_{Retain} = TIA - EIA_{allowable} = (IMP*A_{project}) - EIA_{allowable} \qquad \text{(Equation 2-2)}$$

Where:

A_{Retain} = the drainage area from which runoff must be retained (acres)

TIA = total impervious area (acres)

$EIA_{allowable}$	=	the maximum impervious area from which runoff can be treated and discharged offsite [and not retained onsite] (acres).
IMP	=	imperviousness of project area (%)/100
$A_{project}$	=	the total project area (acres)

Step 5c: Calculate the Volume to be Retained (SQDV)

All Retention BMPs used to render impervious surfaces "ineffective" should be properly sized to retain the volume of water that results from the water quality design storm. The design storm volume, referred to in the TGM as the Stormwater Quality Design Volume (SQDV) shall be calculated using the following four allowable methodologies:

1) The 85th percentile 24-hour runoff event determined as the maximized capture stormwater volume for the area using a 48 to 72-hour draw down time, from the formula recommended in Urban Runoff Quality Management, WEF Manual of Practice No. 23/ASCE Manual of Practice No. 87, (1998); or

2) The volume of annual runoff based on unit basin storage water quality volume to achieve 80 percent or more volume treatment; or

3) The volume of runoff produced from a 0.75 inch storm event; or

4) Eighty (80) percent of the average annual runoff volume using an appropriate public domain continuous flow model [such as Storm Water Management Model (SWMM) or Hydrologic Engineering Center – Hydrologic Simulation Program – Fortran (HEC-HSPF)], using the local rainfall record and relevant BMP sizing and design data.

Note: Examples used throughout the 2011 TGM use the 0.75 inch storm event (Methodology #3).

EXAMPLE 2-1: EIA CALCULATION

Given: 10 acre total project area, 55% impervious, 25% landscaped, 20% undisturbed, percent allowable EIA = 5%.

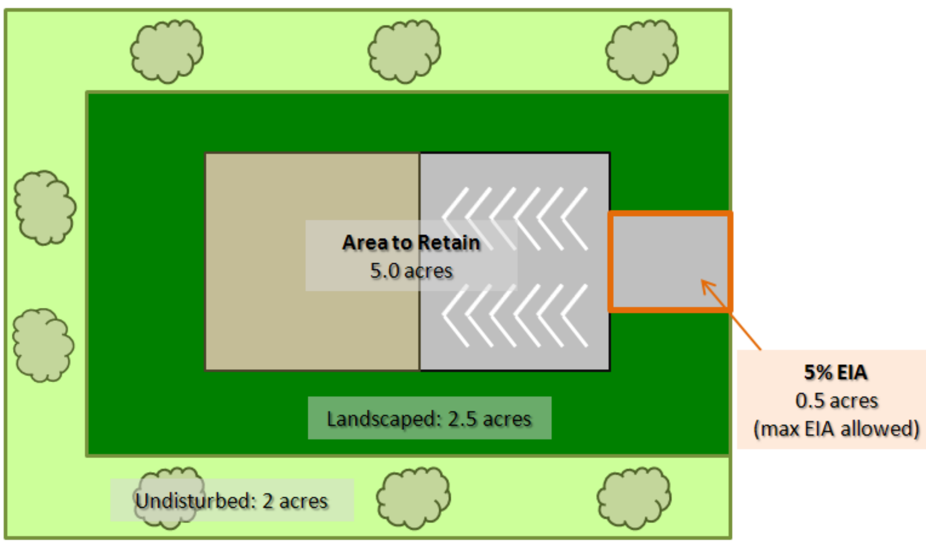
$EIA_{allowable}$ = 10 * 0.05 = 0.5 acres

A_{Retain} = (0.55*10) – 0.5 = 5.0 acres

$A_{treatment}$ = (0.25*10) + 0.5 = 3.0 acres

The maximum EIA allowed for the site is 0.5 acres, from which the generated runoff must be treated prior to discharge, in addition to the runoff from the 2.5 acres landscaped area, up to the design storm volume or flow rate. The runoff volume generated from the remaining 5 acre impervious area (A_{Retain}) must be retained onsite via infiltration, rainwater harvesting, and/or evapotranspiration Retention BMPs.

$A_{treatment}$ equals the EIA allowed for the site plus the landscaped area.



Note: graphic not to scale; for illustration purposes only

The runoff volume that is to be retained onsite should be calculated using Equation 2-3 below:

$$V_{Retain} = C*(0.75/12)*A_{retain} \qquad \text{(Equation 2-3)}$$

Where:

V_{Retain} = the stormwater quality design volume (SQDV) that must be retained onsite (ac-ft)

C	=	runoff coefficient (equals 0.95 for impervious surfaces)
0.75	=	the design rainfall depth (in) [based on SQDV sizing method 3]
A_{Retain}	=	the drainage area from which runoff is retained (acres), calculated using Equation 2-2

EXAMPLE 2-2: RETENTION VOLUME CALCULATION

Given: A_{Retain} = 5.0 acres (from Example 2-1); runoff coefficient (C) = 0.95

$$V_{Retain} = 0.95*(0.75/12)*5.0 \text{ acres} = 0.3 \text{ acre-feet}$$

The project must retain at least 0.3 acre-feet of runoff from impervious surfaces using Retention BMPs.

Step 5d: Select and Size Onsite Retention BMPs to Achieve 5% EIA

The next step is to select and size Retention BMPs, based on the site assessment design, and constraints. Section 3-4 provides guidance on the selection of Retention BMPs. The project must demonstrate disconnection of impervious area such that the 5% EIA requirement is achieved.

Step 5e: Select and Size Biofiltration BMPs to Reduce EIA to ≤5%

Retention BMPs shall be used onsite to the maximum extent practicable. Pretreatment BMPs shall be provided for all infiltration BMPs and other Retention BMPs as needed (see Section 6.1).

New development and redevelopment projects that demonstrate technical infeasibility for reducing EIA to ≤5% using Retention BMPs are eligible to use Biofiltration BMPs to achieve the EIA performance standard.

The project applicant shall demonstrate technical infeasibility by submitting a site-specific analysis conducted and endorsed by a registered professional engineer, geologist, architect, and/or landscape architect. Section 3.2 discusses technical feasibility screening criteria. Projects that cannot demonstrate technical infeasibility shall meet the requirement to reduce EIA to ≤5% using Retention BMPs. Otherwise project applicants must examine other options for meeting the requirements, such as redesigning the site.

Volume-based biofiltration BMPs shall be sized to treat 1.5 times the volume not retained using Retention BMPs.

The onsite biofiltered volume ($V_{Biofilter}$), should be calculated as follows:

$$V_{Biofilter} = (V_{Retain} - V_{Achieved}) * 1.5 \qquad \text{(Equation 2-4)}$$

Where:

$V_{Biofilter}$ = the volume that must be captured and treated in a Biofiltration BMP (ac-ft)

V_{Retain} = the stormwater quality design volume (SQDV) that must be retained (ac-ft) (established in Step 5c)

$V_{Achieved}$ = the volume retained onsite using Retention BMPs (ac-ft)

EXAMPLE 2-3: BIOFILTRATION VOLUME CALCULATION

Given: V_{Retain} = 0.3 ac-ft (from Example 2-2); $V_{Achieved}$ = 0.25 ac-ft

$$V_{Biofilter} = (0.3 – 0.25) * 1.5 = 0.075 \text{ ac-ft}$$

If the project applicant has demonstrated technical infeasibility, the remaining EIA requirement may be met by biofiltering 1.5 times the remaining V_{Retain}. In this case, the Biofiltration BMP must be sized to treat 0.075 ac-ft.

If the project applicant has demonstrated technical infeasibility, the remaining EIA requirement may also be satisfied with flow-based Biofiltration BMPs. Flow-based Biofiltration BMPs shall be sized for the remaining drainage area from which runoff must be retained (A_{Retain}) using the methodology described in Section 2.8, Stormwater Quality Design Flow, with a rainfall intensity that varies with time of concentration for the catchment tributary to the flow-based Biofiltration BMP, according to Table 2-1.

Table 2-1: Flow-Based Biofiltration BMP Design Intensity for 150% Sizing

Time of Concentration, minutes	Design Intensity for 150% Sizing, in/hr
30	0.24
20	0.25
15	0.28
10	0.31
5	0.35

Time of concentration should be determined using the methodology provided in the Ventura County Hydrology Manual.

2.7 Step 6: Alternative Compliance

Certain new development and redevelopment project types are eligible for alternative compliance measures if onsite Retention BMPs and/or Biofiltration BMPs cannot feasibly be used to meet the 5% EIA standard (see Section 3.2). Such projects include:

1) Redevelopment projects (as defined in Section 1.5).

2) Infill projects. Infill projects meet the following conditions:

   a. The project is consistent with applicable general plan designation, and all applicable general plan policies, and applicable zoning designation and regulations;

   b. The proposed development occurs on a project site of no more than five acres substantially surrounded by urban uses;

   c. The project site has no value as habitat for endangered, rare, or threatened species;

   d. Approval of the project would not result in any significant effects relating to traffic, noise, air quality, or water quality; and

   e. The site can be adequately served by all required utilities and public services (modified from State Guidelines § 15332).

3) Smart Growth projects. Smart Growth projects are defined as new development and redevelopment projects that occur within existing urban areas[2] (see maps in Appendix B) designed to achieve the majority of the following principles[3]:

   a. Create a range of housing opportunities and choices;

   b. Create walkable neighborhoods;

   c. Mix land uses;

---

[2] Existing urban areas and corresponding maps in Appendix B are based on the cities' City Urban Restriction Boundaries (CURB) lines and in the case of the unincorporated County, the Existing Community designation. These boundaries are a growth management tool intended to channel growth and protect agricultural and open-space land. The 2011 TGM utilizes existing urban areas (as defined in Appendix B) to provide parameters around eligibility for alternative compliance in two areas: 1) Smart Growth and 2) low income housing projects.

[3] Adapted from the Smart Growth Network's Smart Growth Principles in cooperation with the U.S. Environmental Protection Agency.

d. Preserve open space, natural beauty, and critical areas;

   i. Farmland preservation may also be considered for projects occurring outside existing urban areas (as defined by the Appendix B maps).

e. Provide a variety of transportation choices;

   i. Includes transit oriented development (development located within an average 2,000 foot walk to a bus or train station).[4]

f. Strengthen and direct development towards existing communities (as defined by Appendix B maps); and

g. Take advantage of compact building design.

The City or County Planning Division in which a project is proposed will ultimately determine whether a project meets these Smart Growth criteria.

4) Pedestrian/bike trail projects:

   ✓ Located along side of a road and

   ✓ Where right-of-way width is inadequate for the implementation of Retention and/or Biofiltration BMPs.

5) Agency flood control, drainage, and wet utilities projects:

   ✓ Located within waterbody and is therefore not increasing functional impervious cover; or

   ✓ Located on top of a narrow flood control feature (such as a levee) and space is unavailable for the implementation of Retention and/or Biofiltration BMPs; or

   ✓ Where the integrity of the flood control feature (such as a dam or levee) may be compromised through Retention and/or Biofiltration BMPs (e.g., infiltration of stormwater is not appropriate in a levee).

6) Historical preservation projects:

   ✓ Where the extent of the designated preservation area restricts the amount of land available for the implementation of Retention BMPs.

[4] Calthorpe, P. (1993), "The next American metropolis: Ecology, community, and the American dream", New York: Princeton Architectural Press.

7) Low income housing projects that occur within existing urban areas (as defined by the maps provided in Appendix B):

   - ✓ Where density requirements restrict the amount of land available for the implementation of Retention BMPs and/or

   - ✓ Where project financing constraints restrict the amount of land available for the implementation of Retention BMPs.

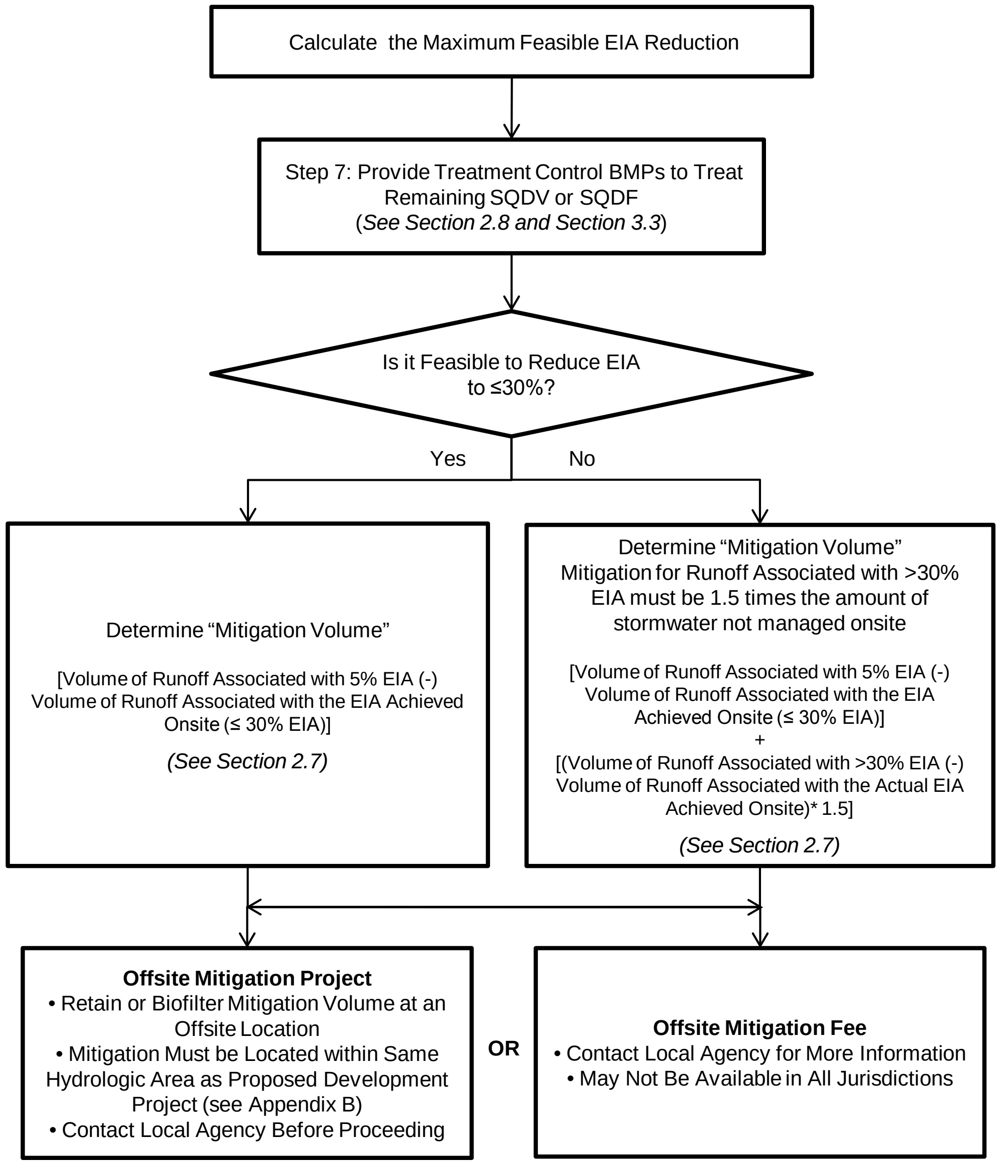


Figure 2-3: Alternative Stormwater Management Control Measures Compliance Decision Flow Chart

Projects in these categories must demonstrate that full compliance with the 5% EIA standard using Retention BMPs and Biofiltration BMPs is infeasible prior to moving to the alternative compliance flowchart (Figure 2-3) and selecting an offsite mitigation alternative. Section 3.2 provides infeasibility criteria.

Stormwater runoff from impervious surfaces and developed pervious surfaces that is not fully retained onsite (up to the SQDV) shall be mitigated using Treatment Control Measures [Chapter 6] selected per the BMP selection process outlined in Section 3.3, in addition to offsite alternative compliance measures.

Alternative compliance may be met through two options:

- Offsite mitigation project; or
- Offsite mitigation fee.

In either case, the Project applicant must contact the local approval agency before proceeding with Alternative Compliance.

Mitigation Volume

Projects requesting alternative compliance must demonstration that EIA has been reduced to the maximum extent practicable. Additionally, the SQDV or SQDF from all directly connected impervious area and the developed pervious project area must be captured and treated within the project site.

Alternative compliance options will be based on the "mitigation volume." The mitigation volume is the difference between the volume of runoff associated with 5% EIA and the volume of runoff associated with the actual EIA achieved onsite less than or equal to 30% (≤30%) EIA. The offsite mitigation requirement for EIA in excess of 30% (>30%) is 1.5 times the amount of stormwater not managed onsite.

Projects Feasible to Reduce EIA to ≤ 30%

1) Determine the volume of runoff that is retained and biofiltered onsite ($V_{Ret/Bio}$), using Equation 2-5 below:

$$V_{Ret/Bio} = (V_{Achieved} + (V_{Biofiltered}/1.5)) \qquad \text{(Equation 2-5)}$$

Where:

$V_{Ret/Bio}$ = the total volume of runoff retained and/or biofiltered onsite using Retention and Biofiltration BMPs

$V_{Achieved}$ = the runoff volume retained onsite using Retention BMPs as calculated in Equation 2-4

$V_{Biofiltered}$ = the runoff volume biofiltered onsite

2) Determine the Mitigation Volume ($V_{Mitigation}$), using Equation 2-6 below:

$$V_{Mitigation} = V_{Retain} - V_{Ret/Bio} \quad \text{(Equation 2-6)}$$

Where:

$V_{Mitigation}$ = the volume of runoff that must be mitigated offsite

V_{Retain} = the SQDV that must be retained onsite per the 5% EIA requirement calculated in Equation 2-3

$V_{Ret/Bio}$ = the total volume of runoff retained and/or biofiltered onsite using Retention and Biofiltration BMPs calculated in Equation 2-5

EXAMPLE 2-4: ≤30% EIA OFFSITE MITIGATION VOLUME CALCULATION

Given: V_{Retain} = 0.3 ac-ft (from Example 2-2); $V_{Retained}$ = 0.25 ac-ft; $V_{Biofiltered}$ = 0.06 ac-ft

1) Calculate volume of runoff retained and biofiltered onsite ($V_{Ret/Bio}$).

$V_{Ret/BioBio}$ = 0.25 + (0.06/1.5) = 0.29 ac-ft [See Equation 2-5]

2) Calculate Mitigation Volume: ($V_{Mitigation}$):

$V_{Mitigation}$ = 0.3– 0.29 = 0.01 acre-feet [See Equation 2-6]

The required offsite mitigation volume is 0.01 ac-ft.

In addition, the SQDV or SQDF from the EIA (0.5 acres) and the developed pervious area (10 acres *25% = 2.5 acres) must be captured and treated in an approved Treatment Control Measure.

SQDV (acre-feet) = C*(0.75/12)*3 acres

OR

SQDF (cfs) = C * 0.20 in/hr * 3 acres

Note: Per Order R4-2010-0108, several options exist to determine the SQDV and SQDF. Examples used throughout the 2011 TGM use the 0.75 inch storm event (SQDV Methodology #3) for the SQDV and 0.2 inches per hour intensity for the SQDF (SQDF Methodology #1). For these examples, the 10-acre project site is assumed to be in a location where the 85th percentile storm event is equal to 0.75 inches.

Projects with EIA > 30%

For the scenario where the effective impervious area of the project is greater than 30% due to infeasibility, the runoff volume associated with the effective impervious area up to 30% must be mitigated offsite at a one-to-one ratio and the runoff volume associated with the effective impervious area greater than 30% must be mitigated off-site at 1.5 times the volume.

1) Determine the area of the impervious portion of the drainage area from which runoff is retained or biofiltered at 30% EIA ($A_{30\%EIA}$), using Equation 2-7 below:

$$A_{30\%EIA} = (IMP*A_{project}) - (30\%*A_{project}) \quad \text{(Equation 2-7)}$$

Where:

$A_{30\%EIA}$	=	the impervious portion of the drainage area from which runoff would have been retained or biofiltered at 30% EIA (acres)
IMP	=	total imperviousness of project area (%)/100
$A_{project}$	=	the total project area (acres)

2) Determine the total volume that would have been retained or biofiltered onsite at 30% EIA ($V_{30\%EIA}$), using Equation 2-8 below:

$$V_{30\%EIA} = C*(0.75/12)*A_{30\%EIA} \qquad \text{(Equation 2-8)}$$

Where:

$V_{30\%EIA}$	=	the stormwater quality design volume (SQDV) retained or biofiltered at 30% EIA (note: for the purposes of this calculation, the biofiltered volume does not include the 1.5 multiplier)
C	=	runoff coefficient [equals o.95 for impervious surfaces]
0.75	=	the design rainfall depth (in) [based on SQDV sizing method 3]
$A_{30\%EIA}$	=	the impervious area from which runoff would have been retained or biofiltered at 30% EIA (acres) [See Equation 2-7]

3) Determine the impervious area from which runoff is actually retained ($A_{ActualEIA}$). This is the total amount of impervious area that drains to properly sized Retention or Biofiltration BMPs.

$$A_{ActualEIA} = (IMP*A_{project}) - (EIA\%*A_{project}) \qquad \text{(Equation 2-9)}$$

Where:

$A_{ActualEIA}$	=	the impervious portion of the drainage area from which runoff is retained or biofiltered using the actual EIA achieved on-site (acres)
IMP	=	total imperviousness of project area (%)/100
$A_{project}$	=	the total project area (acres)
EIA%	=	percent EIA actually achieved on-site

4) Determine the volume that is actually retained onsite ($V_{ActualEIA}$), using Equation 2-10 below:

$$V_{ActualEIA} = C*(0.75/12)*A_{AcutalEIA} \qquad \text{(Equation 2-10)}$$

Where:

$V_{AcutalEIA}$ = the stormwater quality design volume (SQDV) that is retained and/or biofiltered onsite C = runoff coefficient [equals o.95 for impervious surfaces]

0.75 = the design rainfall depth (in) [based on SQDV sizing method 3]

$A_{ActualEIA}$ = the area associated with the Actual EIA achieved onsite, (i.e., the area from which runoff is retained or biofiltered (acres) [See # 3 above]

Determine the Mitigation Volume for 30% EIA using Equation 2-11 below:

$$V_{Mitigation30\%} = V_{Retain} - V_{30\%EIA} \qquad \text{(Equation 2-11)}$$

Where:

$V_{Mitigation30\%}$ = the mitigation volume for Project site with 30% EIA

V_{Retain} = the SQDV that must be retained onsite per the 5% EIA requirement, calculated using Equation 2-3

$V_{30\%EIA}$ = the runoff that would have been retained and/or biofiltered at 30% EIA (note: for the purposes of this calculation, the biofiltered volume does not include the 1.5 multiplier), calculated using Equation 2-8

Determine the Mitigation Volume for >30% (EIA $V_{Mitigation>30\%}$), using Equation 2-12 below:

$$V_{Mitigation>30\%} = (V_{30\%EIA} - V_{ActualEIA})*1.5 \qquad \text{(Equation 2-12)}$$

Where:

$V_{Mitigation>30\%}$ = the mitigation volume for >30% EIA

$V_{30\%EIA}$ = the stormwater quality design volume (SQDV) retained or biofiltered at 30% EIA (note: for the purposes of this calculation, the biofiltered volume does not include the 1.5 multiplier)

$V_{ActualEIA}$ = the stormwater quality design volume (SQDV) that is actually retained and/or biofiltered onsite, calculated using Equation 2-9

Determine the Total Mitigation Volume ($V_{MitigationTotal}$), using Equation 2-13 below:

$$V_{MitigationTotal} = V_{Mitigation>30\%} + V_{Mitigation30\%} \qquad \text{(Equation 2-13)}$$

Where:

$V_{MitigationTotal}$ = the total mitigation volume for 30% EIA

$V_{Mitigation>30\%}$ = the mitigation volume for >30% EIA, calculated using Equation 2-11

$V_{Mitigation30\%}$ = the mitigation volume for 30% EIA calculated using Equation 2-10.

EXAMPLE 2-5: >30% EIA OFFSITE MITIGATION CALCULATION

Given: 40% EIA; 10 acre total project area, 55% impervious, 25% landscaped, 20% undisturbed; runoff coefficient (C) = 0.95; V_{Retain} = 0.3 ac-ft

1) Determine impervious area retained or biofiltered onsite at 30% EIA
   $A_{30\%EIA}$ = ((55/100)*10) – ((30/100)*10) = 2.5 acres [See Equation 2-7]

2) Determine the volume that is retained or biofiltered onsite at 30% EIA
   $V_{30\%EIA}$ = 0.95*(0.75/12)*2.5 = 0.15 ac-ft [See Equation 2-8]

3) Determine the impervious area from which runoff is actually retained
   $A_{ActualEIA}$ = ((55/100)*10) – ((40/100)*10) = 1.5 acres [See Equation 2-9]

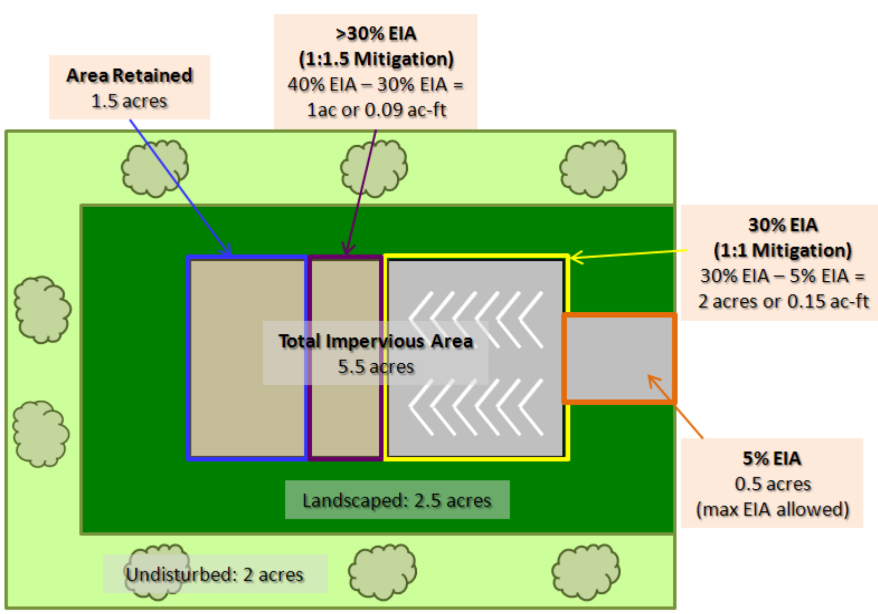
4) Determine the volume that is actually retained or biofiltered onsite
   $V_{ActualEIA}$ = 0.95*(0.75/12)*1.5 = 0.09 ac-ft [See Equation 2-10]

5) Determine Mitigation Volume for 30% EIA
   $V_{Mitigation30\%}$ = 0.3 – 0.15 = 0.15 ac-ft [See Equation 2-11]

6) Determine Mitigation Volume for >30%
   $V_{Mitigation>30\%}$ = (0.15-0.09) *1.5 = 0.09 ac-ft [See Equation 2-12]

7) Determine the Total Mitigation Volume
   $V_{MitigationTotal}$ = 0.15 + 0.09 = 0.24 ac-ft [See Equation 2-13]

The required offsite mitigation volume is 0.24 ac-ft



Selecting Offsite Mitigation Projects

Project applicants may identify offsite mitigation projects. Project applicants are responsible for completing offsite mitigation projects that will achieve equivalent volume and pollutant load reduction using Retention and/or Biofiltration BMPs sized for the mitigation volume. Offsite mitigation projects must adhere to the following criteria:

- Offsite mitigation projects must be located within the same hydrologic area (see map in Appendix B)
- Offsite mitigation projects must be completed as soon as possible and at the latest, within 4 years of the certificate of occupancy for the original project.

Examples of Offsite Mitigation Projects

Mitigation projects should target urbanized areas that were developed without stormwater mitigation. All projects must be approved by the local permitting agency and must adhere to the BMP Selection Criteria presented in Section 3.3 of the 2011 TGM. Potential project types may include:

- Convert a convex parking lot landscaped island into a depressed bioretention area designed to retain parking lot runoff.
- Convert a traditionally-paved parking lot into porous pavement.
- Modify an existing detention pond into a retention pond.
- Install bioretention in bump-outs, in parkways, or in roadway medians.
- Install bioretention in sidewalk areas to infiltrate roof, sidewalk, and/or roadway runoff. Sidewalks must be wide enough to permit foot traffic around bioretention area.
- Incorporate infiltration BMPs into landscaped areas that collect runoff from impervious surfaces.
- Regional BMPs.

Offsite Mitigation Fee

In some cases, Alternative Compliance may be achieved through an Offsite Mitigation Fee. A list of offsite mitigation projects available for funding will be identified by the Approval Agencies. Applicants should contact their local Approval Agency for more information. The Offsite Mitigation Fee may not be available in all jurisdictions.

2.8 Step 7: Apply Treatment Control Measures

Stormwater runoff from EIA and developed pervious surfaces shall be mitigated using Retention BMPs, Biofiltration BMPs, or Treatment Control Measures [Chapter 6] selected per the BMP selection process outlined in Section 3.3. Biofiltration BMPs and Treatment Control Measures may be sized to meet the Stormwater Quality Design Volume (SQDV) or the Stormwater Quality Design Flow (SQDF). Treatment Control Measures should be designed in adherence with the guidance provided in Section 6 of the 2011 TGM in order to assure a level of pollutant removal comparable to those listed in Attachment "C" of Order R4-2010-0108 (also provided in Appendix D.1).

Projects that are eligible for Offsite Mitigation must still provide treatment for all impervious surfaces and developed pervious areas using Treatment Control Measures sized to meet the SQDV or SQDF on site. Treatment Control Measures must be selected per the BMP selection process outlined in Section 3.3.

Stormwater Quality Design Volume (SQDV)

Volume-based Treatment Control Measures must be sized to capture and treat the runoff volume from the water quality design storm. The SQDV shall be calculated using the following four allowable methodologies:

1) The 85th percentile 24-hour runoff event determined as the maximized capture stormwater volume for the area using a 48 to 72-hour draw down time, from the formula recommended in Urban Runoff Quality Management, WEF Manual of Practice No. 23/ASCE Manual of Practice No. 87, (1998); or

2) The volume of annual runoff based on unit basin storage water quality volume to achieve 80 percent or more volume treatment; or

3) The volume of runoff produced from a 0.75 inch storm event; or

4) Eighty (80) percent of the average annual runoff volume using an appropriate public domain continuous flow model [such as Storm Water Management Model (SWMM) or Hydrologic Engineering Center – Hydrologic Simulation Program – Fortran (HEC-HSPF)], using the local rainfall record and relevant BMP sizing and design data.

The allowable design storm calculation methodology for Treatment Control Measures, per Order R4-2010-0108, is determined by the total project disturbed land area, as summarized in Table 2-2 below.

Table 2-2: Allowed Design Storm Methodology Based on Project Size

Project Size (Disturbed Land Area[1])	Allowed Design Storm Methodology
Less than 5 acres	(1), (2), (3), or (4)
5 acres - 50 acres	(1), (2), or (4)
More than 50 acres	(4)

[1] "Disturbed Area" means any area that is altered as a result of land disturbance, such as clearing, grading, grubbing, stockpiling or excavation.

Instructions for calculating the SQDV based on method (3), the volume of runoff produced from a 0.75 inch storm event, are provided below. Instructions for calculating the SQDV for methods (1), (2), and (4) are provided in Appendix E. Note that Biofiltration BMPs must be sized to treat 1.5 times the volume not retained using Retention BMPs as indicated in Step 5e.

Calculation Procedure

1) Determine the area from which runoff must be retained or captured and treated ($A_{project}$).

2) Determine the runoff coefficient (C), using Equation 2-13 below:

$$C = 0.95*imp + C_p\,(1-imp) \qquad \text{(Equation 2-13)}$$

Where:

C	=	runoff coefficient (equals 0.95 for impervious surfaces)
imp	=	impervious fraction of watershed
C_p	=	pervious runoff coefficient, determined based on soil type using table below [see Ventura County Hydrology Manual (2006)]:

Table 2-3: Ventura Soil Type Pervious Runoff Coefficients

Ventura Soil Type (Soil Number)	C_p value
1	0.15
2	0.10
3	0.10
4	0.05
5	0.05
6	0
7	0

3) Determine the stormwater runoff design volume (SQDV), using Equation 2-14 below:

$$SQDV = C^*(0.75/12)^*\ A_{project} \qquad \text{(Equation 2-14)}$$

Where:

SQDV	=	the stormwater quality design volume (acre-feet)
C	=	runoff coefficient, calculated by Equation 2-13
0.75	=	the design rainfall depth (in) [based on sizing method (3)]Atrib
$A_{project}$	=	drainage area of the tributary catchment (acres)

Stormwater Quality Design Flow (SQDF)

For the purposes of the 2011 TGM, instructions for calculating the SQDF based on method (1), the flow of runoff produced from a rainfall event equal to at least 0.2 inches per hour intensity, are provided below. Instructions for calculating the SQDF for methods (2), and (3) are provided in Appendix E. Note that flow-based Biofiltration BMPs used to achieve 5% EIA must be sized per the design intensity specified in Table 2-1.

Calculation Procedure

1) Determine the drainage area from which the flow-based BMP will be receiving runoff ($A_{project}$).

2) Calculate the runoff coefficient (C), using Equation 2-13.

3) Calculate the SQDF using Equation 2-15 below:

$$SQDF = C*I*A_{project} \quad \text{(Equation 2-15)}$$

Where:

SQDF	=	flow in cubic feet per second (cfs)
C	=	runoff coefficient, calculated by Equation 2-13 above
I	=	average rainfall intensity (inches/hour) for a duration equal to the time of concentration of the watershed [equal to 0.2 in/hr for method (1); see also Table 2-1:]
$A_{project}$	=	drainage area of the tributary catchment (acres)

2.9 Step 8: Continue Project Design Process: Flood Control and Hydromodification Requirements

The project applicant should continue with the design process to address additional requirements including flood control and hydromodification control criteria.

Step 8a: Flood Control Requirements

Applicants shall comply with Ventura County and local approval agency regulations on floodplain and floodway management.

Step 8b: Hydromodification (Flow/Volume/Duration) Control Criteria

Projects meeting the applicability criteria contained in Section 4.E.II of Order R4-2010-0108 (presented in Section 1.5 of the 2011 TGM) are required to implement hydrologic control measures to prevent accelerated erosion and to protect stream habitat in downstream natural drainage systems. Natural drainage systems are defined as unlined or unimproved (not engineered) creeks, streams, rivers and their tributaries.

Exemptions

The following new development and redevelopment projects are exempt from the hydromodification control criteria:

1) Single-family structures, unless such projects disturb one acre or more of land or create, add, or replace 10,000 square feet or more of impervious surface area.

2) All projects that disturb less than one acre.

3) Projects that are replacement, maintenance, or repair of an Agency's existing flood control facility, storm drain, or transportation network.

4) Redevelopment projects in existing urban areas [see maps in Appendix B] that do not increase the effective impervious area or decrease the infiltration capacity of pervious areas compared to the pre-developed condition.

5) Projects that have any increased discharge directly or via a storm drain to a sump, lake, area under tidal influence, into a waterway that has a 100-year peak flow (Q100) of 25,000 cubic feet per second (cfs) or more, or other receiving water that is not susceptible to hydromodification impacts.

6) Projects that discharge directly or via a storm drain into concrete or improved (not natural) channels (e.g., rip rap, sackcrete, etc.), which, in turn, discharge into receiving water that is not susceptible to hydromodification impacts (as in #5 above).

Hydromodification Control Measures

The purpose of Hydromodification Control Measures is to minimize changes in post-development stormwater runoff discharge rates, velocities, and durations by maintaining within a certain tolerance, the project's pre-developed stormwater runoff flow rates and durations.

Hydromodification Control Measures may include onsite, subregional, or regional Hydromodification Control Measures, Retention BMPs, or stream restoration measures. Preference must be given to onsite Retention BMPs and Hydromodification Control Measures. In-stream restoration measures may not adversely affect the beneficial uses of natural drainage systems.

The Southern California Stormwater Monitoring Coalition (SMC) is developing a regional methodology to eliminate or mitigate the adverse impacts of hydromodification as a result of urbanization, including hydromodification assessment and management tools. The Program will develop and implement watershed-specific Hydromodification Control Plans (HCPs) after the completion of the SMC study. Until the completion of the HCPs, the Interim Hydromodification Control Criteria, described below, apply to applicable, non-exempt new development and redevelopment projects.

Interim Hydromodification Control Criteria

1) Projects disturbing less than 50 acres must comply with the Stormwater Management Standards contained in the 2011 TGM (i.e., a combination of Retention BMPs, Biofiltration BMPs, and/or Treatment Control Measures).

2) Projects disturbing 50 acres or greater must develop and implement a Hydromodification Analysis Study (HAS) that demonstrates that post development conditions are expected to approximate the pre-developed erosive

effect of sediment transporting flows in receiving waters. The HAS must lead to the incorporation of project design features intended to approximate, to the extent feasible, an Erosion Potential value of 1, or any alternative value that can be shown to be protective of the natural drainage systems from erosion, incision, and sedimentation that can occur as a result of flow increases from impervious surfaces and damage stream habitat in natural drainage systems. The methodology for calculating Erosion Potential is provided in Appendix E of Order R4-2010-0108. Project proponents must work with their local permitting authority to ensure that the HAS is correctly prepared.

2.10 Step 9: Develop Maintenance Plan

The Ventura Countywide Stormwater Quality Management Program (Program) requires the submittal of a Maintenance Plan and execution of a Maintenance Agreement with the owner/operator of any stormwater control that requires maintenance including Site Design Principles and Techniques (Section 4); Source Control Measures (Section 5; and Retention BMPs, Biofiltration BMPs, and Treatment Control Measures (Section 6). Maintenance Plans must include guidelines for how and when inspection and maintenance should occur for each control. Section 7 and Appendices H and I provide additional information and guidance on compliance with maintenance requirements.

3 SITE ASSESSMENT AND BMP SELECTION

3.1 Assessing Site Conditions and Other Constraints

Assessing a site's potential for implementation of Retention BMPs, Biofiltration BMPs, and Treatment Control Measures requires both the review of existing information and the collection of site-specific measurements. Available information regarding site layout and slope, soil type, geotechnical conditions, and local groundwater conditions should be reviewed as discussed below. In addition, soil and infiltration testing should be conducted to determine if stormwater infiltration is feasible and to determine the appropriate design infiltration rates for infiltration-based treatment BMPs.

Site Conditions

Topography

The site's topography should be assessed to evaluate surface drainage and topographic high and low points, as well as to identify the presence of steep slopes that qualify as Hillside Locations. All of these conditions have an impact on what type of Retention BMPs, Biofiltration BMPs, and Treatment Control Measures will be most beneficial for a given project site. Stormwater infiltration is more effective on level or gently sloping sites. Flows on slopes steeper than 15% may runoff as surface flows, rather than infiltrate into the ground. On hillsides, infiltrated runoff may daylight or resurface a short distance downslope, which could cause slope instability depending on the soil or geologic conditions. See the Geotechnical Considerations section below.

Soil Type and Geology

The site's soil types and geologic conditions should be determined to evaluate the site's ability to infiltrate stormwater and to identify suitable, as well as unsuitable, locations for infiltration-based BMPs (e.g., infiltration basins and trenches, bioretention without an underdrain, permeable pavement, and drywells). Using the Soil Survey completed by the Soil Conservation Service (SCS) (now identified as the Natural Resource Conservation Service [NRCS]) of the U. S. Department of Agriculture in April 1970, soils in Ventura County were grouped into seven hydrologically homogeneous families [see Ventura County Hydrology Manual (2006); also see Appendix B]. Two families were assigned to each of the NRCS Hydrologic Soil Groups A, B, and C; while only one family was considered appropriate for NRCS Hydrologic Soil Group D [for further information, see http://soils.usda.gov/]:

- Group A soils are typically sands, loamy sands, or sandy loams. Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep and well to excessively drained sands or gravels and have a high rate of water transmission. Ventura County soil numbers 6 and 7 are Group A soils.

- Group B soils are typically silty loams or loams. They have a moderate infiltration rate when thoroughly wetted and consist chiefly of moderately deep to deep and moderately well to well drained soils with moderately fine to moderately coarse texture. Ventura County soil numbers 4 and 5 are Group B soils.

- Group C soils are typically sandy clay loams. They have low infiltration rates when thoroughly wetted, consist chiefly of soils with a layer that impedes downward movement of water, and/or have moderately fine to fine soil structure. Ventura County soil numbers 2 and 3 are Group C soils.

- Group D soils are typically clay loams, silty clay loams, sandy clays, silty clays, or clays. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with high swelling potential, permanent high water table, claypan or clay layer at or near the surface, and/or shallow soils over nearly impervious material. Ventura County soil number 1 is a Group D soil.

Infiltration-based BMPs should be feasible in areas mapped with Ventura County Soil Numbers 4 through 7. If site-specific data is available, then soils with infiltration rates of 0.5 in/hr or greater are considered feasible for infiltration. Infiltration-based BMPs should not be designed for sites mapped with Ventura County Soil Numbers 1 through 3 (unless site specific testing is performed and shows an infiltration rate greater than 0.5 in/hr) or with site-specific infiltration rates less than 0.5 in/hr.

Locations where soils are mapped with Ventura Hydrology Manual Soil Number 3, or where a site-specific analyses show that the soils have an infiltration rate of 0.3 to 0.5 inches per hour, and no other infiltration-related infeasibility criteria apply, shall use a Bioinfiltration BMP (or Rainwater Harvesting). Bioinfiltration is an adaption of the Bioretention with an Underdrain BMP in which the underdrain is raised above the gravel storage layer in order to promote infiltration but allow release of biotreated runoff to the storm drain when infiltration capacity is reached.

Early identification of soil types throughout the project footprint can reduce the number of test pit investigations and infiltration tests needed. Early identification reduces the number of potential test sites to locations with those that are most likely to be amenable to infiltration. Guidance for conducting test pit investigations and infiltration tests is provided in Appendix C.

Project applicants should review available geologic or geotechnical reports on local geology to identify relevant features such as depth to bedrock, rock type, lithology, faults, and hydrostratigraphic or confining units. These geologic investigations may also identify shallow water tables and past groundwater issues that are important for BMP design (see below).

Groundwater Considerations

Site groundwater conditions should be considered prior to Retention BMP, Biofiltration BMP, and Treatment Control Measure siting, selection, sizing, and design. The depth to groundwater beneath the project during the wet season may preclude infiltration, since five feet of separation to the seasonal high ground water level and mounded groundwater level is required. Depth to seasonal high groundwater level shall be estimated as the average of the annual minima (i.e., the shallowest recorded measurements in each water year, defined as October 1 through September 30) for all years on record. If groundwater level data are not available or not considered to be representative, seasonal high groundwater depth can be determined by redoximorphic analytical methods combined with temporary groundwater monitoring for November 1 through April 1 at the proposed project site.

In areas with known groundwater pollution, infiltration may need to be avoided, as it could contribute to the movement or dispersion of groundwater contamination. Areas with known groundwater impacts include sites listed by the Los Angeles Regional Water Quality Control Board's Leaking Underground Storage Tanks (LUST) program and Site Cleanup Program (SCP). The California State Water Resources Control Board maintains a database of registered contaminated sites through their 'Geotracker' Program. Registered contaminated sites can be identified in the project vicinity when the site address is typed into the "map cleanup sites" field.

Mobilization of groundwater contaminants may also be of concern where contamination from natural sources is prevalent (e.g., marine sediments, selenium rich groundwater, to the extent that data is available). Infiltration on sites with contaminated soils or groundwater that could be mobilized or exacerbated by infiltration is not allowed, unless a site-specific analysis determines the infiltration would be beneficial. A site-specific analysis may be conducted where groundwater pollutant mobilization is a concern to allow for infiltration-based BMPs.

Research conducted on the effects of stormwater infiltration on groundwater by Pitt et al. (1994) indicate that the potential for contamination due to infiltration is dependent on a number of factors, including the local hydrogeology and the chemical characteristics of the pollutants of concern. Chemical characteristics that influence the potential for groundwater impacts include high mobility (low absorption potential), high solubility fractions, and abundance of pollutants in urban runoff. As a class of constituents, trace metals tend to adsorb onto soil particles and are filtered out by the soils. This has been confirmed by extensive data collected beneath stormwater detention/retention ponds in Fresno (conducted as part of the Nationwide Urban Runoff Program (Brown & Caldwell, 1984)) that showed that trace metals tended to be adsorbed in the upper few feet in the bottom sediments. Bacteria are also filtered out by soils. More mobile and soluble pollutants, such as chloride and nitrate, have a greater potential for impacting groundwater.

Where soils have very high infiltration rates, groundwater quality may be impacted by infiltration BMPs. Prior to the use of infiltration basins and subsurface infiltration BMPs in areas with high infiltration rates, consult with the local regulatory agencies to identify if unconfined aquifers are located beneath the project to determine the appropriateness of infiltration-based BMPs. In areas underlain by unconfined aquifers with designated beneficial groundwater uses (e.g. drinking water supply), the application of infiltration BMPs should be limited to those that provide significant pretreatment to ensure groundwater is protected from pollutants of concern.

Geotechnical Considerations

Water infiltration can cause geotechnical issues, including: (1) settlement through collapsible soil, (2) expansive soil movement, (3) slope instability, and (4) increased liquefaction hazard. Stormwater infiltration temporarily raises the groundwater level near the infiltration facility, such that the potential geotechnical conditions are likely to be of greatest significance near the infiltration area and decrease with distance. A geotechnical investigation should be performed for the infiltration facility to identify potential geotechnical issues and geological hazards that may result from infiltration.

In general, infiltration-based BMPs must be set back from building foundations or steep slopes. Increased water pressure in soil pores reduces soil strength. Decreased soil strength can make foundations more susceptible to settlement and slopes more susceptible to failure. Recommendations for each site should be determined by a licensed geotechnical engineer based on soils boring data, drainage patterns, and the current requirements for stormwater treatment. Implementing the geotechnical engineer's requirements is essential to prevent damage from increased subsurface water pressure on surrounding properties, public infrastructure, sloped banks, and even mudslides.

Collapsible Soil

Typically, collapsible soil is observed in sediments that are loosely deposited, separated by coatings or particles of clay or carbonate, and subject to saturation. Stormwater infiltration will result in a temporary rise in the groundwater elevation. This rise in groundwater could change the soil structure by dissolving or deteriorating the intergranular contacts between the sand particles, resulting in a sudden collapse, referred to as hydrocollapse. This collapse phenomenon generally occurs during the first saturation episode after deposition of the soil, and repeated cycles of saturation are not likely to result in additional collapse. It is important to evaluate the potential for hydrocollapse during the geotechnical investigation.

The magnitude of hydrocollapse is proportional to the thickness of the soil column where infiltration is occurring. In most instances, the magnitude of hydrocollapse will be small. Regardless, the geotechnical engineer should evaluate the potential effects of hydrocollapse from large infiltration facilities on nearby structures and roadways. Typically, a network of surface settlement monuments is installed around the infiltration site, along adjacent roadways, and in neighboring developments to evaluate if hydrocollapse has occurred. These monuments are typically monitored prior to infiltrating stormwater, monthly during the first year of operation of the facility, then yearly thereafter for a period of approximately five years.

Expansive Soil

Expansive soil is generally defined as soil or rock material that has a potential for shrinking or swelling under changing moisture conditions. Expansive soils contain clay minerals that expand in volume when water is introduced and shrink when the water is removed or the material is dried. When expansive soil is present near the ground surface, a rise in groundwater from infiltration activities can introduce moisture and cause these soils to swell. Conversely, as the groundwater surface falls after infiltration, these soils will shrink in response to the loss of moisture in the soil structure. The effects of expansive soil movement (swelling and shrinking) will be greatest on near surface structures such as shallow foundations, roadways, and concrete walks. Basements or below-grade parking structures can also be affected as additional loads are applied to the basement walls from the large swelling pressures generated by soil expansion. A geotechnical investigation should identify if expandable materials are present near the proposed infiltration facility, and if they are, evaluate if the infiltration will result in wetting of these materials. See Appendix B, Map B-14 (expansive soil potential map).

Slopes

Slopes near the infiltration facility can be affected by the temporary rise in groundwater. The presence of a water surface near a slope can substantially reduce the stability of the slope from a dry condition. A groundwater mounding analysis should be performed to evaluate the rise in groundwater around the facility. If the computed rise in groundwater approaches nearby slopes, then a separate slope stability evaluation should be performed to evaluate the implications of the temporary groundwater surface. The geotechnical and groundwater mounding evaluations should identify the duration of the elevated groundwater and assign factors of safety consistent with the duration (e.g., temporary or long-term conditions).

Liquefaction

Seismically-induced soil liquefaction is a phenomenon in which saturated granular materials, typically possessing low to medium density, undergo matrix rearrangement, develop high pore water pressure, and lose shear strength due to cyclic ground motions induced by earthquakes. This rearrangement and strength loss is followed by a reduction in bulk volume. Manifestation of soil liquefaction can include loss of bearing capacity for foundations, surface settlements, and tilting in level ground. Soil liquefaction can also result in instabilities and lateral spreading in embankments and areas of sloping ground.

Saturation of the subsurface soils above the existing groundwater table may occur as a result of stormwater infiltration. A groundwater mounding analysis should also evaluate the duration of mounding, as a lengthy duration or long-term rise in groundwater will need to be considered in the evaluation of liquefaction. If the granular soils are sufficiently dense, it is unlikely that liquefaction will be of concern, regardless of the groundwater mounding. If analyses indicate that the potential for liquefaction may be increased from stormwater infiltration, then the analyses will need to evaluate the liquefaction-induced settlement of structures, lateral spreading, and other surface manifestations. See Appendix B, Map B-14 (liquefaction potential map).

Managing Offsite Drainage

Locations and sources of offsite run-on onto the site should be identified early in the design process. Offsite drainage should be considered when determining appropriate BMPs so that drainage can be managed. Concentrated flows from offsite drainage may cause extensive erosion, if not properly conveyed through or around the project site or otherwise managed. By identifying the locations and sources of offsite drainage, the volume of water running onto the site may be estimated and factored into the siting and sizing of onsite BMPs. Vegetated swales or storm drains may be used to intercept, divert, and convey offsite drainage through or around a site to prevent flooding or erosion that might otherwise occur.

Existing Utilities

Existing utility lines that are onsite will limit the possible locations of certain BMPs. For example, infiltration BMPs should not be located near utility lines where the increased amount of water could damage the utilities. Stormwater should be directed away from existing underground utilities. Project designs that require the relocation of existing utilities should be avoided, if possible.

Environmentally Sensitive Areas

The presence of Environmentally Sensitive Areas (ESAs) may limit the siting of certain BMPs. ESA's are typically delineated by and fall under the regulatory oversight of state or federal agencies such as the U.S. Army Corp of Engineers (USACE), California Department of Fish and Game, U.S. Fish and Wildlife Service, or the California Environmental Protection Agency. BMPs should be selected and sited to avoid adversely affecting an ESA. The Ventura County ESA map (ESA as defined in Order R4-2010-0108) is provided in Appendix B or may be obtained from the local permitting authority.

3.2 Technical Feasibility Screening

To use biofiltration BMPs and alternative compliance measures, the project applicant should demonstrate that compliance with the requirement to reduce EIA to ≤5% using Retention BMPs is technically infeasible by submitting a site-specific hydrologic and/or design analysis conducted and endorsed by a registered professional engineer and/or geologist. Projects seeking to use alternative compliance measures must demonstrate EIA has been reduced to the maximum extent practicable. Project applicants should contact their local Approval Agency to determine if additional infeasibility criteria apply. Technical infeasibility may result from conditions including the following:

1) Locations where seasonal high groundwater or mounded groundwater beneath an infiltration BMP is within 5 feet of the bottom of the infiltration BMP.

2) Locations on the project site where soils are mapped with Ventura Hydrology Manual Soil Numbers 1-2 or site-specific analyses show that the soils have an infiltration rate less than 0.3 inches per hour. Locations where soils are mapped with Ventura Hydrology Manual Soil Number 3, or where a site-specific analyses show that the soils have an infiltration rate of 0.3 to 0.5 inches per hour, and no other infiltration-related infeasibility criteria apply, shall use a Bioinfiltration BMP or Rainwater Harvesting (if feasible) to achieve the 5% EIA requirement.

3) Locations on the project site within 100 feet of a groundwater well used for drinking water, non-potable wells, drain fields, and springs; locations less than 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project; and locations less than eight feet from building foundations or an alternative setback established by the geotechnical expert for the project.

4) Locations where pollutant mobilization is a documented concern, unless a site-specific analysis determines that infiltration would not be detrimental. Portions of brownfield development sites may be eligible for alternative compliance where pollutant mobilization is a concern.

5) Locations with potential geotechnical hazards established by the geotechnical professional for the project.

6) Projects with high-risk areas such as service/gas stations, truck stops, and heavy industrial sites, unless a site-specific evaluation demonstrates that:

    - Treatment is provided to address pollutants of concern, and/or

    - High risks areas are isolated from stormwater runoff or infiltration areas with little chance of spill migration.

7) Locations where reduction of surface runoff may potentially impair beneficial uses of the receiving water as documented in a site-specific study (e.g., California Environmental Quality Act (CEQA) analysis) or watershed plan.

8) Location where an increase in infiltration over natural conditions could potentially cause impairments to downstream beneficial uses, such as change of seasonality of ephemeral washes, as confirmed through a site-specific study.

9) Green roofs are not required to be considered for all project locations and types; this evapotranspiration BMP is considered optional subject to the approval of the permitting authority.

10) Projects that do not provide sufficient demand for harvested stormwater such that the system provides 80% capture with a 72 hour drawdown time considering all "allowable and reliable demand."

    a. Allowable and reliable demand is defined as the rate of use of harvested water under average wet season conditions (November through March), from sources meeting the following criteria:

        - The use is permitted by building codes and health codes without requiring disinfection and fine filtration.

        - The use is reliable on a seasonal basis, such that the lowest weekly demand on an average annual basis is no less than 2/7th of the wet season average. *Intent: Under worst-case conditions, the demand should still be sufficient to use the entire tank volume within a week.*

        - Where a reliable use is present on the site that is not permitted by building codes and/or health codes, a variance has been sought to allow use without disinfection and fine filtration.

        - The use does not conflict with mandatory use of reclaimed water. It is assumed that uses do not conflict unless water balance calculations are provided to demonstrate the contrary.

        - The estimated use rates are consistent with requirements for low water use landscaping requirements under local and statewide ordinance (including California Assembly Bill 1881).

11) BMPs that are not allowable per current federal, state or local codes are considered infeasible. Local codes will be updated by mid-2012 as required in Order R4-2010-0108 (Provision III.D).

12) The following project types where the density and/or nature of the project would create significant difficulty for compliance with the requirement to reduce EIA to ≤5%:

    a. Redevelopment projects (as defined in Section1.5).

    b. Infill projects that meet the following conditions:

        i. The project is consistent with applicable general plan designation, and all applicable general plan policies, and applicable zoning designation and regulations;

ii. The proposed development occurs on a project site of no more than five acres substantially surrounded by urban uses;

iii. The project site has no value as habitat for endangered, rare, or threatened species;

iv. Approval of the project would not result in any significant effects relating to traffic, noise, air quality, or water quality; and

v. The site can be adequately served by all required utilities and public services (modified from State Guidelines § 15332).

c. Smart Growth projects, which are defined as new development and redevelopment projects that occur within existing urban areas (see maps in Appendix B) designed to achieve the majority of the following principles :

i. Create a range of housing opportunities and choices;

ii. Create walkable neighborhoods;

iii. Mix land uses;

iv. Preserve open space, natural beauty, and critical areas;

1. Farmland preservation may also be considered for projects occurring outside existing urban areas (as defined by the Appendix B maps).

v. Provide a variety of transportation choices;

vi. Includes transit oriented development (development located within an average 2,000 foot walk to a bus or train station).

vii. Strengthen and direct development towards existing communities (as defined by Appendix B maps); and

viii. Take advantage of compact building design.

The City or County Planning Division in which a project is proposed will ultimately determine whether a project meets these Smart Growth criteria.

13) Pedestrian/bike trail projects:

✓ Located along side of a road and

✓ Where right-of-way width is inadequate for the implementation of Retention and/or Biofiltration BMPs.

14) Agency flood control, drainage, and wet utilities projects:

- ✓ Located within waterbody and is therefore not increasing functional impervious cover; or
- ✓ Located on top of a narrow flood control feature (such as a levee) and space is unavailable for the implementation of Retention and/or Biofiltration BMPs; or
- ✓ Where the integrity of the flood control feature (such as a dam or levee) may be compromised through Retention and/or Biofiltration BMPs (e.g., infiltration of stormwater is not appropriate in a levee).

15) Historical preservation projects:

- ✓ Where the extent of the designated preservation area restricts the amount of land available for the implementation of Retention BMPs.

16) Low income housing projects that occur within existing urban areas (as defined by the maps provided in Appendix B):

- ✓ Where density requirements restrict the amount of land available for the implementation of Retention BMPs and/or
- ✓ Where project financing constraints restrict the amount of land available for the implementation of Retention BMPs.

Determining Maximum Volume Feasibly Infiltrated and/or Biofiltered

Site conditions and constraints may make it infeasible to fully retain stormwater to achieve ≤ 5% EIA using Retention BMPs. In such cases, stormwater runoff must be retained to the maximum extent practicable and then the remaining volume must be multiplied by 1.5 and biofiltered to the maximum extent practicable. If SQDV still remains, it may be addressed in an alternative compliance program. This section provides narrative and numeric criteria for determining the “maximized” volume for Infiltration BMPs and Biofiltration BMPs. The term “maximized” refers to the volume that is determined, on a case-by-case basis, to be consistent with the maximum extent practicable standard.

Criteria for Maximizing Infiltration Volume

Volume can be considered to be maximized in infiltration BMPs when all of the following conditions are met, or when adjustments to the site/BMP plan to meet any one of these criteria results in achievement of the ≤5% EIA performance standard:

1) BMPs are designed to the maximum depth allowed by design standards, but are not required to exceed the depth that infiltrates within 48 hours at the design percolation rate. *Explanation: Deeper BMPs provide more volume per footprint*

area, therefore it is more feasible to retain stormwater in deeper BMPs than shallower BMPs. However, because of the nature of sequential storms in Southern California, the volume provided in excess of that which drains within 48 hours provides significantly diminishing value.

2) All practicable methods are employed to enhance the design percolation rate, including:

   - Use of soil amendments to native soil below infiltration BMPs, and
   - Provision of pretreatment to reduce the allowable factor of safety, and
   - Additional site investigation to reduce uncertainty in infiltration rate and allow the use of a lower factor of safety.

3) Good site practices have been integrated to provide the maximum pervious area feasible for infiltration BMPs, and infiltration BMPs have been configured to make use of this area. Table 3-1 provides recommended percentages of a site, by project type, that should be feasible to dedicate to infiltration BMPs (where technically feasible) within pervious areas. If the project has not provided this portion of the project site for infiltration BMPs (where technically feasible), an attempt should be made to improve site design to provide more pervious area until it is either infeasible to provide more pervious area or EIA is reduced to ≤5%. The minimum percent of parking lot pavement area considered feasible to dedicate to permeable pavement (where technically feasible) is 20%; this does not apply to parking lots that anticipate heavy truck traffic such as truck stops and heavy industrial areas. The criteria provided in Table 3-1 are guidance; each project will be individually evaluated by the local permitting authority to determine if good site practices have been integrated into the project to provide the maximum pervious area feasible for siting infiltration BMPs.

Criteria for Maximizing Biofiltration Volume

Biofiltration BMPs can be used downstream of a Retention BMP that has been "maximized" (e.g., a planter box treating overflow from a cistern) or can be designed to provide both "maximized" retention and "maximized" biofiltration in the same BMP (e.g., a bioretention area with an underdrain, where retention volume is provided in a gravel layer or other subsurface reservoir below the underdrain).

Volume can be considered to be maximized in Biofiltration BMPs when all of the following conditions are met, or when adjustments to the site design and BMP plan to meet any one of these criteria results in achievement of the ≤5% EIA performance standard:

1) Drain time and/or treatment rate of the Biofiltration BMP is consistent with design guidance contained in Section 6 of the 2011 TGM.

2) Good site practices have been integrated to provide the maximum area feasible for Biofiltration BMPs, and BMPs have been configured to make use of this area. Table 3-1 provides recommended percentages of a site that are feasible to be dedicated to Biofiltration BMPs by project type. If the project has not provided these portions of the project site for siting Biofiltration BMPs, an attempt should be made to improve site design to provide more area until it is either infeasible to provide more area or EIA is reduced to ≤5%. The criteria provided in Table 3-1 are guidance; each project will be individually evaluated by the local permitting authority to determine if good site practices have been integrated into the project to provide the maximum pervious area feasible for siting Biofiltration BMPs.

If a Biofiltration BMP also includes a retention component (e.g., storage volume in a swale in amended soil below the surface discharge elevation or storage below the underdrain of a bioretention area), the maximized retention volume is determined as the volume of water that can be infiltrated or evapotranspired within 48 hours after the Biofiltration BMP has emptied. This criterion should be used to establish the depth of the retention layer (i.e., the depth of amended soil below the swale or the size of the storage below underdrains in the bioretention area).

Table 3-1: Recommended Criteria for Percent of Site Feasible to Dedicate to BMPs

Project Type		Percent of Site[1]
New Development	SF/MF Residential < 7 du/ac	10
	SF/MF Residential 7 – 18 du/ac	7
	SF/MF Residential > 18 du/ac	5
	Mixed Use, Commercial, Institutional/Industrial w/ FAR < 1.0	10
	Mixed Use, Commercial, Institutional/Industrial w/ FAR 1.0 – 2.0	7
	Mixed Use, Commercial, Institutional/Industrial w/ FAR > 2.0	5
	Podium (parking under > 75% of project)	3
	Projects with zoning allowing development to lot lines	2
	Transit Oriented Development	5
	Parking	5

Project Type		Percent of Site[1]
Redevelopment	SF/MF Residential < 7 du/ac	5
	SF/MF Residential 7 – 18 du/ac	4
	SF/MF Residential > 18 du/ac	3
	Mixed Use, Commercial, Institutional/Industrial w/ FAR < 1.0	5
	Mixed Use, Commercial, Institutional/Industrial w/ FAR 1.0 – 2.0	4
	Mixed Use, Commercial, Institutional/Industrial w/ FAR > 2.0	3
	Podium (parking under > 75% of project)	2
	Projects with zoning allowing development to lot lines	1
	Transit Oriented Development	3
	Projects in Historic Districts	3

Key: SF = Single Family, MF = Multi Family, du/ac = dwelling units per acre, FAR = Floor Area Ratio = ratio of gross floor area of building to gross lot area.

[1] If subsurface BMPs are used, dedicated area may have other surface land uses which do not structurally impact the subsurface BMP (see INF-6: Proprietary Infiltration).

3.3 Treatment Control Measure Selection Guidance

Treatment Control Measure selection criteria contained in Order R4-2010-0108 include the following:

- Treatment Control Measures shall be selected based on the primary class of pollutants likely to be discharged from the project (e.g., metals from an auto repair shop).

- For projects that discharge to an impaired waterbody and whose discharges contain the pollutant causing impairment, the project shall select Treatment Control Measures from the top three performing BMP categories, or alternative BMPs that are designed to meet or exceed the performance of the highest performing BMP, for the pollutant causing impairment.

Primary Class of Pollutants

Pollutants in stormwater runoff are typically related to land use activities, which means that the proposed project's site uses provide some indication of the pollutants that will be generated in the site's runoff. Table 3-2 identifies pollutants of concern based on typical land use activities that may be present on a project site.

Table 3-2: Land Uses and Associated Pollutants

Class of Pollutant	Potential Land Use and Activities Sources
Sediment (TSS and Turbidity)	Streets, driveways, roads, landscaped areas, construction activities, soil erosion (channels and slopes)
Nutrients	Landscape fertilizers, atmospheric deposition, automobile exhaust, soil erosion, animal waste, detergents
Metals/Metalloids	Automobiles, bridges, atmospheric deposition, industrial areas, soil erosion, metal surfaces, combustion processes
Pesticides	Landscaped areas, roadsides, utility right-of-ways
Organic Materials/ Oxygen Demanding Substances	Landscaped areas, animal wastes, industrial wastes
Oil and Grease/ Organics Associated with Petroleum	Roads, driveways, parking lots, vehicle maintenance areas, gas stations, automobile emissions, restaurants
Bacteria and Viruses	Lawns, roads, leaky sanitary sewer lines, sanitary sewer cross-connections, animal waste (domestic and wild), septic systems, homeless encampments, sediments/biofilms in stormwater conveyance system
Trash and Debris (Gross Solids and Floatables)	Commercial areas, roadways, schools, trash receptacles/storage/disposal

Adapted from US EPA, 1999 (Preliminary Data Summary of Urban Stormwater BMPs)

Impaired Waterbodies

When designated beneficial uses of a particular receiving water body are being compromised by water quality for a specific or multiple pollutants, Section 303(d) of the CWA requires identifying and listing that water body as "impaired".

Table 3-3 below lists the categories of pollutants and specific pollutants that are included on the 2010 303(d) list for Ventura County. Project proponents should consult the most recent 303(d) list to identify whether the project's receiving waterbody is listed as impaired. The most recent 303(d) list is located on the State Water Resources Control Board website (click on water issues/programs/water quality assessment).

Table 3-3: Ventura County 2010 303(d)-listed Water Quality Pollutants

Class of Pollutant	Specific Pollutants		
Sediment (TSS and Turbidity)	Sedimentation/Siltation		
Nutrients	Ammonia Nitrate and Nitrite Nitrate Nitrogen	Organic Enrichment/ Low Dissolved Oxygen	Algae Eutrophic
Metals/Metalloids	Boron Copper Copper, Dissolved	Lead Mercury Nickel	Selenium Zinc
Pesticides	ChemA (tissue) Chlordane Chlordane (tissue & sediment) Chlordane (tissue) Chlorpyrifos Chlorpyrifos (tissue) DDT DDT (sediment) DDT (tissue & sediment)	DDT (tissue) Diazinon Dieldrin Dieldrin (tissue) Organophosphorous Pesticides Toxaphene Toxaphene (tissue & sediment) Toxaphene (tissue)	
Trash and Debris (Gross Solids and Floatables)	Trash and Debris		
Other Organics	PCBs		
Bacteria and Viruses	Coliform Bacteria	Indicator Bacteria	
Salinity	Chloride		
Toxicity	Sediment Toxicity	Toxicity	
Miscellaneous	pH	Scum/Foam - unnatural	Sulfates

Once the classes of pollutants likely to be discharged from the project have been identified for projects that do not discharge to an impaired waterbody, any Treatment Control Measures listed in Table 3-4 that addresses the primary pollutant class may be selected. If more than one pollutant class is identified, then sediment shall be the primary pollutant class.

For projects that discharge to an impaired waterbody and whose discharges contain the pollutant causing impairment, the project shall select Treatment Control Measures from the top three BMPs listed for that class of pollutant in Table 3-4, or alternative BMPs that are designed to meet or exceed the performance of the highest performing Treatment Control Measure, for the pollutant causing impairment. Many receiving water impairments are due to legacy pollutants from past land use activities (e.g., DDT from historical farming or PCBs from historical industrial activities), where the primary sources are contaminated soils and sediment. For these pollutants, site clean-up, erosion and sediment controls during construction, slope stabilization measures, and placement of impervious surfaces will address the legacy pollutants.

Table 3-4: Treatment Control Measures for Addressing Pollutants of Concern

Class of Pollutant	Recommended BMPs (in Order of Performance)
Sediment	1. Retention BMPs (Infiltration, Rainwater Harvesting, and Evapotranspiration BMPs) 2. Any of the following BMPs(equivalent performance): a. Biofiltration BMPs b. Wet Detention Basin c. Constructed Wetland d. Sand Filter/Cartridge Media Filter 3. Dry Extended Detention Basin
Metals / Metalloids	1. Retention BMPs (Infiltration, Rainwater Harvesting, and Evapotranspiration BMPs) 2. Any of the following BMPs (equivalent performance): a. Constructed Wetland b. Biofiltration BMPs c. Wet Detention Basin d. Sand Filter/Cartridge Media Filter 3. Dry Extended Detention Basin
Nutrients[1]	1. Retention BMPs (Infiltration, Rainwater Harvesting, and Evapotranspiration BMPs) 2. Any of the following BMPs (equivalent performance): a. Bioinfiltration b. Wet Detention Basin c. Constructed Wetland 3. Any of the following BMPs (equivalent performance): a. Biofiltration BMPs 4. Any of the following (equivalent performance): a. Sand Filter/Cartridge Media Filter b. Dry Extended Detention Basin
Pesticides[2]	1. Source controls, erosion controls 2. Retention BMPs (Infiltration, Rainwater Harvesting, and Evapotranspiration BMPs) 3. Any of the following BMPs (equivalent performance): a. Biofiltration BMPs b. Wet Detention Basin c. Constructed Wetland d. Sand Filter/Cartridge Media Filter 4. Dry Extended Detention Basin

Class of Pollutant	Recommended BMPs (in Order of Performance)
Pathogens	1. Retention BMPs (Infiltration, Rainwater Harvesting, and Evapotranspiration BMPs) 2. Any of the following BMPs (equivalent performance): a. Bioretention with Underdrain b. Wet Detention Basins c. Proprietary Biofiltration 3. Sand Filter/Cartridge Media Filter
Trash and Debris	1. Gross Solids Removal BMPs (should be combined with a Retention BMP, Biofiltration BMP, or Treatment Control Measure) 2. Any Retention BMP, Biofiltration BMP, or Treatment Control Measure designed to incorporate a trash capture device (e.g., a trash screen)

[1]Performance is based on removal of nitrogen compounds. For performance of BMPs in removing phosphorous, see sediment pollutant class as they are largely associated with particulates.
[2]Performance data is not available for this pollutant class, but as they are largely associated with particulates, BMP selection should be similar to the sediment pollutant class.

An analysis of Biofiltration BMP and Treatment Control Measure performance from the ASCE International Stormwater BMP Database [1999-2008] is provided in Appendix D. These performance data summaries are occasionally revised. Updated analyses of Biofiltration BMP and Treatment Control Measure performance may be found on the ASCE International Stormwater BMP Database website. The 2011 TGM assumes that BMPs adhering to the design guidance provided in Section 6 will have a level of pollutant removal performance comparable to those listed in Attachment C in Order R4-2010-0108 (also provided in Appendix D.1).

Proprietary BMPs should meet or exceed the performance standards listed in Attachment C in Order R4-2010-0108 and provided in Appendix D.

The data contained in the Stormwater BMP Database indicate that wet detention basins, constructed wetlands, sand filters, and biofilters are among the best performing BMPs for the typical pollutants of concern in urban runoff. This conclusion is consistent with the treatment processes typically provided by these BMP types (e.g., filtration, sedimentation, adsorption, and biological processes).

Wet detention basins (wetponds) and constructed wetlands are attractive solutions both from a treatment process and observed performance perspective. However, these systems require significant base flow to maintain their permanent pools and to avoid creating stagnant conditions and vector concerns. Therefore, these BMPs are often infeasible in locations where water conservation during dry weather is a significant concern. If a regional Treatment Control Measure is desired, infiltration basins and dry extended detention basins may be more feasible in Ventura County. However, these BMPs may need additional treatment train components (e.g., pre- or post-treatment) to adequately address the entire list of pollutants of concern and provide reliable and consistent performance, in addition to significant space requirements. BMP designs for each pollutant category that incorporate dense vegetation and promote extended contact with or filtration through soils are encouraged, consistent with the BMP selection prioritization requirements in Order R4-2010-0108.

Consideration of Site-Specific Conditions

Ultimately, Retention BMPs, Biofiltration BMPs, and Treatment Control Measures have to be constructed at a physical location and site-specific conditions should be considered during the BMP selection process. Site constraints such as steep slopes, poor draining soils, high ground water tables, unstable or contaminated soils and several other factors can preclude the implementation of certain kinds of Retention BMPs, Biofiltration BMPs, and Treatment Control Measures or design options. Therefore, site-specific conditions must be considered when selecting specific BMPs or Treatment Control Measures to implement. Once candidate BMPs or Treatment Control Measures have been chosen, the selection process should consider the site assessment results for soil characteristics, slopes, groundwater proximity, etc. Table 3-5 below provides general guidance for designers regarding site limitations for the different Retention BMPs, Biofiltration BMPs, and Treatment Control Measures.

Table 3-6 below provides general guidance for designers regarding capital and operation costs for the different Retention BMPs, Biofiltration BMPs, and Treatment Control Measures. BMP costs can also be estimated using the Water Environment Research Foundation (WERF) BMP and LID Whole Life Cost Models. These models are set of spreadsheet tools that help users identify and combine capital costs and ongoing maintenance expenditures in order to estimate whole life costs for stormwater management. The models provide a framework for calculating capital and long-term maintenance costs of individual Retention BMPs, Biofiltration BMPs, and Treatment Control Measures. Models are included for retention ponds, extended detention basins, vegetated swales, permeable pavement, green roofs, large commercial cisterns, and bioretention. Online PDF of user's guide and spreadsheet tools are located here: http://www.werf.org/AM/Template.cfm?Section=Research_Profile&Template=/CustomSource/Research/PublicationProfile.cfm&id=SW2R08.

Table 3-5: BMP Site Suitability Considerations

Important Note to Users: This table should be used to provide general BMP comparisons only and should not replace an evaluation performed by a qualified water quality professional.

BMP	Site Suitability Considerations			
	Tributary Area (Acres) [1]	Site Slope (%)	Depth to Seasonally High or Mounded Groundwater (ft)	Soil Number
Infiltration BMPs: INF-1: Infiltration Basin INF-2: Infiltration Trench INF-3: Bioretention INF-4: Drywell INF-6: Proprietary Infiltration	< 5	< 7[2]	> 5	Not suitable in Soil Numbers 1, 2, and 3 unless percolation testing shows the infiltration rate is greater than 0.5 in/hr
INF-5: Permeable Pavement	< 5	< 5[2,5]	> 2 with underdrains; > 5 without underdrains	Underdrains should be provided for Soil Numbers 1, 2, and 3
ET-1: Green Roof	Equal to roof tributary area	N/A	N/A	N/A
BIO-1: Bioretention with Underdrain	< 5	< 15; planter boxes are generally more suitable for steep slopes[2,3]	> 2 with underdrains; > 5 without underdrains	Underdrains should be provided for Soil Numbers 1, 2, and 3
BIO-2: Planter Box	< 1	< 15[4]	> 2	Any
BIO-3: Vegetated Swale	< 5	< 10 site slope; 0.5 to 6 longitudinal slope of swale [2,3]	> 2 with underdrains; > 5 without underdrains	Any[3]

BMP	Site Suitability Considerations			
	Tributary Area (Acres) [1]	Site Slope (%)	Depth to Seasonally High or Mounded Groundwater (ft)	Soil Number
BIO-4: Vegetated Filter Strip	< 2	< 4 site slope; 2 to 6 longitudinal slope of strip[2]	> 2	Any
BIO-5: Proprietary Biotreatment Devices	The site suitability requirements for specific proprietary devices must be provided by the manufacturer and should be verified by independent sources or assessed by a qualified water quality professional.			
TCM-4: Sand Filter	< 10	< 15[4]	> 2	Any
TCM-5: Cartridge Media Filters	The site suitability requirements for specific proprietary devices must be provided by the manufacturer and should be verified by independent sources or assessed by a qualified water quality professional.			
PT-1: Hydrodynamic Devices	The site suitability requirements for specific proprietary devices must be provided by the manufacturer and should be verified by independent sources or assessed by a qualified water quality professional.			
PT-2: Catch Basin Inserts				

[1] Tributary area is the area of the site draining to the BMP. Tributary areas provided here should be used as a general guideline only. Tributary areas can be larger or smaller as appropriate.

[2] If site slope exceeds that specified or if the system is within 200 ft from the top of a hazardous slope or landslide area (on the uphill side), a geotechnical investigation analysis and report addressing slope stability shall be prepared by a licensed civil engineer. In addition, for swales, if the longitudinal slope exceeds 6%, check dams should be provided.

[3] If system is located within 50 feet of a sensitive steep slope (on the uphill side), within 10 feet from a structure, has a longitudinal slope less than 1.5% (swales), or has poorly drained soils (e.g., silts and clays), underdrains should be incorporated.

[4] If system is fully contained, includes an underdrain system, and overflows to a stormwater conveyance system, then slopes can exceed 15%.

[5] If a gravel base is used for storage of runoff: (1) slopes should be restricted to 0.5% (steeper grades reduce storage capacity) and (2) underdrains should be used if within 50 feet of a sensitive steep slope.

[6] Setbacks apply to systems without underdrains.

Table 3-6: BMP Cost Considerations

BMP Type	Relative Expense[4] (cost/ac-ft[1] or cost/cfs[2])	Construction Costs (per cubic feet)[3,4]	Typical Cost[3] ($/BMP)	Typical Cost[3] Application	Annual Maintenance Cost (% of Construction)[3,4]	Notes
Infiltration Trench	Not included	$4- $50	$45,000	5-ac Commercial Site (65% Impervious)	5%-20%	
Infiltration Basin	$	$1.30 - $18	$15,000	5-ac Commercial Site (65% Impervious)	1% -10%	
Bioretention	Not included	$3- $5.30	$60,000	5-ac Commercial Site (65% Impervious)	5%- 7%	Cost of plants varies. Maintenance costs comparable to cost of typical landscaping.
Swale	$$	$0.25-$0.50	$3,500	5-ac Residential Site (35% Impervious)	5%- 7%	
Filter Strip	$$	$0.00- $1.30	$0- $9,000	5-ac Residential Site (35% Impervious)	$350/ acre/ year (about $0.01/square foot/ year)	
Extended Detention Basin	$$$	$0.50- $1.00	Not included		3 to 6%	Costs vary widely. One 0.3 ac-ft basin was recorded to have cost $160,000[5] $3,132 Annual maintenance costs for per Caltrans[5]
Wet Ponds	$$$	$0.50- $1.00	Not included		3 to 6%	$17,000 Annual maintenance costs for one Caltrans pond[5]
Constructed Wetland	$$$$	$0.60 – $1.25	$125,000	50-Acre Residential Site (35% Impervious)	2%	
Sand Filter	$$$$	$3 - $6	$35,000- $70,000	5-Acre Commercial Site (65% Impervious)		

[1] Volume based BMPs

[2] Flow based BMPs

[3] EPA, 1999. Preliminary Data Summary of Urban Storm Water Best Management Practices. Part D, Cost and Benefits Analysis. http://water.epa.gov/scitech/wastetech/guide/stormwater/index.cfm#report

[4] CASQA, 2003. New Development and Redevelopment Handbook

[5] Figures from Caltrans studies cited in CASQA BMP Handbook.

4 SITE DESIGN PRINCIPLES AND TECHNIQUES

4.1 Introduction

The primary objective of the Site Design Principles and Techniques is to reduce the hydrologic and water quality impacts associated with land development. The benefits derived from this approach include**:**

- Reduced size of downstream Treatment Control Measures and conveyance systems;
- Reduced pollutant loading to onsite Treatment Control Measures and receiving streams; and
- Reduced hydraulic impact on receiving streams.

Site Design Principles and Techniques include the following design features and considerations:

- Site planning;
- Protect and restore natural areas;
- Minimize land disturbance;
- Minimize impervious cover;
- Apply Low Impact Development best management practices (LID BMPs) at various scales: and
- Implement Integrated Water Resource Management Practices.

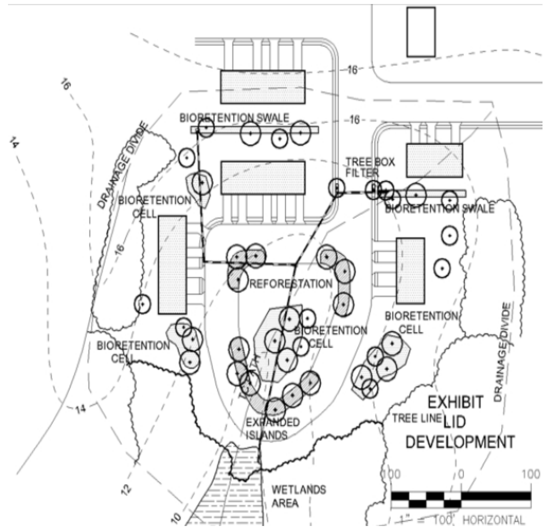
The Site Design Principles and Techniques described in this section are required to be considered for all new development and redevelopment projects subject to conditioning and approval for the design and implementation of post-construction stormwater management control measures (as defined in Section 1.5). They are not required if the project proponent demonstrates to the satisfaction of the City or County that the particular measures are not applicable to the proposed project, or the project site conditions make it infeasible to implement the site design control measure in question. The applicability of specific controls outlined within this section should be confirmed with the local government.

Detailed descriptions and design criteria for each of the Site Design Principles and Techniques are presented in the following section.

4.2 Site Planning

Purpose

LID requires a holistic approach to site design and stormwater management. As such, planners, developers, architects, and engineers should reconsider conventional approaches to stormwater management. The use of site planning techniques presented here will generate a more hydrologically functional site, help to maximize the effectiveness of Retention BMPs, and integrate stormwater management throughout the site.



LID BMPs Integrated within Site Planning Process

Low Impact Development Center, Inc.

Design Criteria

The following criteria should be considered during the early site planning stages:

1) Retention BMPs should be considered as early as possible in the site planning process. Hydrology should be an organizing principle that is integrated into the initial site assessment planning phases.

2) Project applicants should anticipate and plan for the space requirements of Retention and Biofiltration BMPs. Table 4-1 provides general rules of thumb for BMP space requirements.

3) Site planning should use a multidisciplinary approach that includes planners, engineers, landscape architects, and architects at the initial phases of the project.

4) Individual Retention BMPs should be distributed throughout the project site and may influence the configuration of roads, buildings, and other infrastructure.

5) The project must demonstrate disconnection of impervious surface such that the 5% EIA requirement is achieved. If fully meeting the 5% EIA requirement using Retention BMPs is not technically feasible, the project must still utilize Retention BMPs to the maximum extent practicable.

6) Consider flood control early in the design stages. Even sites with Retention BMPs will still have runoff that occurs during large storm events. Look for opportunities to simultaneously address flood control requirements and the requirement to reduce EIA to ≤5% presented in Section 2.

7) Consider the use of alternative building materials instead of conventional materials for new construction and renovation. Several studies have indicated that metal used as roofing material, flashing, or gutters can leach metals into the environment. Avoid the use of roofing, gutters, and trim made of copper and galvanized (zinc) roofs, gutters, chain link fences and siding.

8) Consider 2010 Green Building Code requirements during the site planning stages.

Table 4-1: Rule of Thumb Space Requirements for BMPs[5]

BMP Type	% of Contributing Drainage Area
Infiltration	3 to 10
Rainwater Harvesting (Cistern)	0 to 10
Evapotranspiration (Green Roof)	1 to 1 ratio of impervious cover treated
Biofiltration	3 to 5
Dry Extended Detention Basin	1 to 3
Wet Detention Basin	1 to 3
Sand Filters	0 to 5
Cartridge Media Filter	0 to 5

[5] Modified from Schueler, T., D. Hirschman, M. Novotney, and J. Zielinski. 2007. Urban Stormwater Retrofit Practices. Manual 3 in the Urban Subwatershed Restoration Manual Series. Center for Watershed Protection. Ellicott City, MD.

4.3 Protect and Restore Natural Areas

Purpose

Each project site possesses unique topographic, hydrologic and vegetative features, some of which are more suitable for development than others. Sensitive areas that should be protected and/or restored include streams and their buffers, floodplains, wetlands, steep slopes, and high permeability soils. Additionally, slopes can be a major source of sediment and should be properly protected and stabilized.

Locating development on the least sensitive portion of a site and conserving naturally vegetated areas can minimize environmental impacts in general and stormwater runoff impacts in particular.

Stream Buffer

Larry Walker Associates

Design Criteria

If applicable and feasible for the given site conditions, the following site design features or elements are required and should be included in the project site layout, consistent with applicable General Plan and Local Area Plan policies:

1) Identify and cordon off streams and their buffers, floodplains, wetlands, and steep slopes.

2) Reserve areas with high permeability soils for either open space or Infiltration BMPs.

3) Incorporate existing trees into site layout.

4) Identify areas that may be restored or revegetated either during or post-construction.

5) Identify and avoid and/or stabilize areas susceptible to erosion and sediment loss.

6) Concentrate or cluster development on the least-sensitive portions of a site, while leaving the remaining land in a natural undisturbed state.

7) Slopes must be protected from erosion by safely conveying runoff from the tops of slopes.

    - Slopes should be vegetated by first considering use of native or drought-tolerant species.

- Slope protection practices must conform to local permitting agency erosion and sediment control standards and design standards. The design criteria described in this section are intended to enhance and be consistent with these local standards.

8) Limit clearing and grading of native vegetation at the project site to the minimum amount needed to build lots, allow access, and provide fire protection.

9) Maintain existing topography and existing drainage divides to encourage dispersed flow.

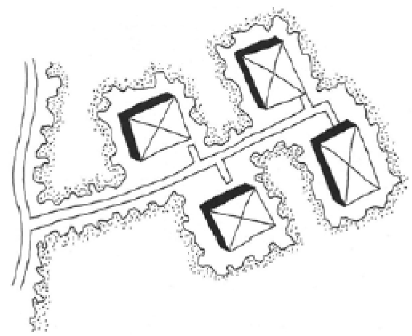
10) Maximize trees and other vegetation at each site by planting additional vegetation, clustering tree areas, and promoting the use of native and/or drought-tolerant plants.

11) Promote natural vegetation by using parking lot islands and other landscaped areas. Integrate vegetated BMPs within parking lot islands and landscaped areas.

4.4 Minimize Land Disturbance

Purpose

This control works to protect water quality by preserving some of the natural hydrologic function of the site. By designing a site layout to preserve the natural hydrology and drainageways on the site, it reduces the need for grading the disturbance of vegetation and soils (GSMM, 2001). By siting buildings and impervious surfaces away from steep slopes, drainageways, and floodplains, it limits the amount of grading, clearing and distance and reduces the hydrologic impact. This site design principle has most applicability in greenfield settings, but opportunities may exist in redevelopment and infill projects.



Minimized Clearing and Grading

Greenfield et al., 1991

Existing soils may contain organic material and soil biota that are ideal for storing and infiltrating stormwater. Clearing, grading, and heavy equipment can remove and compact existing soils and, therefore, limit their infiltrative capacity. The design criteria presented below are not intended to supersede compaction requirements associated with building codes.

Design Criteria

1) Delineate and flag the development envelope for the site. Delineating and flagging the development envelope includes a clear indication of the development envelope on the site plan and physical demarcation in the field which can be accomplished using temporary orange construction fencing or flagging. The development envelope can be established by identifying the minimum area needed to build lots; allow access and provide fire protection; and protect and buffer sensitive features such as streams, floodplains, steep slopes and wetlands. Concentrate buildings and paved areas on the least permeable soils, with the least intact habitats.

2) Plan clearing and grading to minimize the compaction of infiltrative soils.

3) Restrict equipment access and storage of construction equipment to the development envelope.

4) Restrict storage of construction equipment within the development envelope.

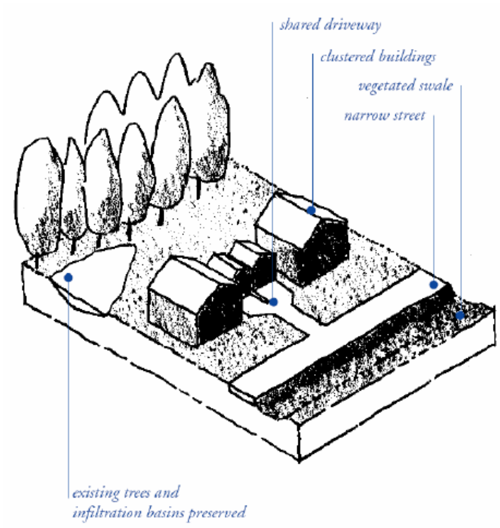
5) Avoid the removal of existing trees and valuable vegetation, as feasible.

6) Consider soil amendments to restore permeability and organic content especially for infill and redevelopment projects to avoid soil disturbance.

4.5 Minimize Impervious Cover

Purpose

The potential for the discharge of pollutants in stormwater runoff from a project site increases as the percentage of impervious area within the project site increases because impervious areas increase the volume and rate of runoff flow. Pollutants deposited on impervious areas tend to be easily mobilized and transported by surface water runoff. Minimizing impervious area through site design is an important means of minimizing stormwater pollutants of concern. In addition to the environmental and aesthetic benefits, a highly pervious site may allow reduction in the size of downstream conveyance and treatment systems, yielding savings in development costs. Reducing impervious area is the most cost effective way of minimizing the effective impervious area (EIA) requirement.



Impervious Cover Minimization

BASMAA, Start at the Source

Design Criteria

Local permitting agency building and fire codes and ordinances determine some aspects of site design. These design strategies are intended to enhance and be consistent with these local codes and ordinances. Minimizing impervious surfaces at every possible opportunity requires integration of many small strategies. Suggested strategies for minimizing impervious surfaces through site design include the following:

1) Use minimum allowable roadway cross sections, driveway lengths, and parking stall widths and lengths.

2) Minimize or eliminate the use of curbs and gutters, and maximize the use of Retention BMPs, where slope and density permit.

3) Use two-track/ribbon alleyways/driveways or shared driveways.

4) Include landscape islands in cul-de-sac streets. Consider alternatives to cul-de-sacs to increase connectivity.

5) Reduce the footprints of building and parking lots. Building footprints may be reduced by building taller.

6) Use permeable pavement to accommodate overflow parking (if overflow parking is needed).

7) Cluster buildings and paved areas to maximize pervious area.

8) Maximize tree preservation or tree planting.

9) Avoid compacting or paving over soils with high infiltration rates (see Minimize Land Disturbance).

10) Use pervious pavement materials where appropriate, such as modular paving blocks, turf blocks, porous concrete and asphalt, brick, and gravel or cobbles.

11) Use grass-lined channels or surface swales to convey runoff instead of paved gutters (see Vegetated Swale in Section 6).

12) Build more compactly in infill and redevelopment site to avoid disturbing natural and agricultural lands. Per capita impacts can be significantly reduced by building more compactly in infill and redevelopment areas.

4.6 Apply LID at Various Scales

Purpose

LID is a decentralized approach to stormwater management that works to mimic the natural hydrology of the site by retaining rainfall onsite. In order to realize the full benefits of water quality protection and runoff volume reduction, LID should be integrated and considered at the regional and watershed scale and the site scale.

Design Criteria

Regional/Watershed

1) Consider Density: Low density development has a greater water resource impact than compact growth on a watershed scale. Higher density development uses less land and produces less impervious cover per capita than low density development (USEPA, 2006). Developments should consider higher densities, but should still adhere to density levels as specified within local zoning requirements.

2) Identify and Preserve Contiguous Open Space: Large contiguous areas of open space can act as a flood control, have an ecological benefit, serve as a buffer for streams and rivers, and provide recreational opportunities (EPA, 2004). Applicants should look for opportunities to link open space preservation with regional open space preservation efforts (such as Save Open Space and Agricultural Resources).

3) Make use of Previously Developed Sites: Redevelopment of existing sites replace impervious cover with impervious cover, reduces the need for greenfield development, and makes use of existing infrastructure.

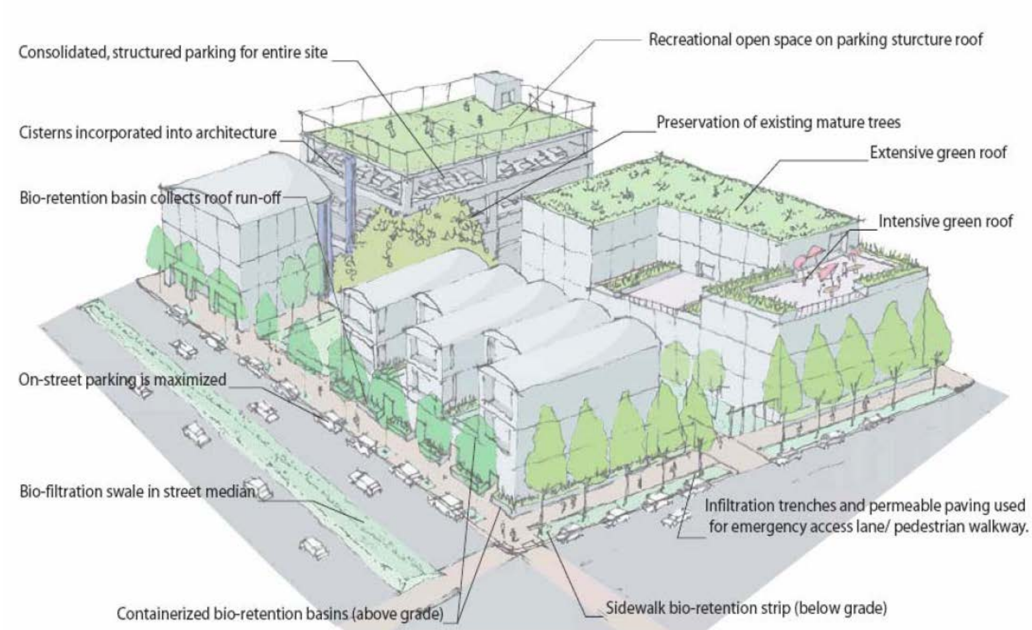
4) Locate Compact Development within Close Proximity to Mass Transit: This maximizes transportation choices, reduces the number of automobile trips, and lessens the water quality impacts associated with transportation and low-density sprawl.

Site

The following design criteria should be considered at the site level in addition to the principles and techniques discussed earlier in this section (e.g., Minimize Impervious Cover).

1) Maintain and Restore Natural Flowpaths for Runoff: Site buildings and impervious surfaces away from steep slopes, drainageways, and floodplains to reduce the amount of necessary clearing and grading and maintain the pre-development hydrology's time of concentration.

2) Maximize Use of Existing Impervious Cover: Assess and take advantage of opportunities to use existing impervious surfaces at the site level to reduce runoff at a watershed scale.



LID BMPs Considered at Various Scales

C. Anderson, Sustainable Urbanism

3) Design Public Spaces and Common Areas to Minimize Stormwater Runoff: Public spaces and common areas can serve as community gathering places but are often composed of impervious cover (e.g., courtyards primarily made up of concrete) (EPA, 2004). Design public spaces and common areas to accommodate both people and stormwater management.

4) Compact Project Design: Compact project design reduces the amount of impervious cover per capita, increases walkability, and decreases water quality impacts associated with transportation. Concentrating development on one portion of the site reduces the amount of lawn, provides more opportunities to preserve open space, and maintains and restores natural flow paths. Additionally, compact design can reduce street and driveway length and as a result, can help to reduce the imperviousness associated with development.

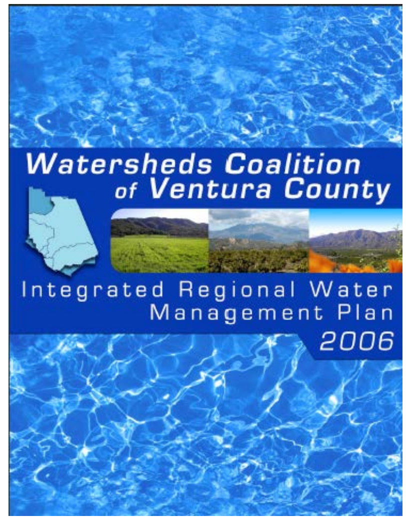
5) Encourage Use of Multiple Modes of Transportation: In addition to density and compact design, additional aspects of site design may encourage the use of multiple modes of transportation:

- Bicycle and pedestrian-friendly streets;
- Well connected sidewalks and streets; and
- Mixed uses that encourage walking.

4.7 Implement Integrated Water Resource Management Practices

Purpose

Integrated Water Resource Management (IWRM) is a process which promotes the coordinated development and management of water, land, and related resources. Order R4-2010-0108 promotes the use of IWRM to help guide the selection of BMPs that conserve water, recharge groundwater, provide recreational opportunities and serve as multiple purpose parks and preserve open space.



Integrated Regional Water Management Plan

Ventura County

Many of the concepts of IWRM are documented in the County's Integrated Regional Water Management Plan (IRWMP). The IRWMP is the product of an intensive stakeholder process and addresses multiple water resource management goals including improved water supply reliability, water recycling, water conservation, recreation and access, flood control, wetlands enhancement and creation, and environmental and habitat protection (Watershed Coalition of Ventura County, 2006).

Design Criteria

The goals of the 2011 TGM and the new development and redevelopment requirements contained within Order R4-2010-0108, complement the goals of the IRWMP. Development projects should strive to select BMPs that meet the following multiple objectives (Watershed Coalition of Ventura County, 2006):

1) Conserve and Augment Water Supplies: Identify and evaluate the opportunities to recharge groundwater and increase water use efficiency. This can be accomplished through infiltration of stormwater runoff and selection of drought-tolerant landscaping.

2) Protect People, Property and the Environment from Adverse Flooding Impacts: Identify opportunities to utilize BMPs that provide both water quality and water quantity benefits. Provide and maintain setbacks from streams and rivers.

3) Protect and Restore Habitat and Ecosystems in Watersheds: Implement the practices identified in Protect and Restore Natural Areas to integrate habitat and stormwater goals. Landscaping selection for stormwater management practices may also further encourage and attract wildlife.

4) Provide Water-related Recreational, Public Access and Educational Opportunities: Integrate recreation and stormwater management by creating multi-functional BMPs and designing courtyards and open spaces that accommodate both people and stormwater runoff. Consider providing educational signs for BMPs located in public spaces, where appropriate.

5 SOURCE CONTROL MEASURES

5.1 Introduction

Source Control Measures are low-technology practices designed to prevent pollutants from contacting stormwater runoff and prevent discharge of contaminated runoff to the storm drainage system. This section addresses site-specific, structural-type Source Control Measures consisting of specific design features or elements. Non-structural type Source Control Measures; such as good housekeeping and employee training, are not included in the 2011 TGM. The project applicant can consult the California Industrial Best Management Practice Manual for this type of practice (SWQTF, 1993). The governing stormwater agency may require additional Source Control Measures not included in the 2011 TGM for specific pollutants, activities, or land uses.

This section describes control measures for specific types of sites or activities that have been identified as potential significant sources of pollutants in stormwater. Each of the measures specified in this section should be implemented in conjunction with appropriate non-structural Source Control Measures to optimize pollution prevention.

The measures addressed in this section apply to both stormwater and non-stormwater discharges. Non-stormwater discharges are the discharge of any substance, such as process wastewater, to the storm drainage system or water body that is not composed entirely of stormwater. Stormwater that is mixed or commingled with other non-stormwater flows is considered non-stormwater. Discharges of stormwater and non-stormwater to the storm drainage system or a water body may be subject to local, state, or federal permitting prior to discharge. The appropriate agency should be contacted prior to any discharge. Discuss the matter with the stormwater staff if you are uncertain as to which agency should be contacted.

Some of the measures presented in this section require connection to the sanitary sewer system. It is prohibited to connect and discharge to the sanitary sewer system without prior approval or obtaining the required permits. Contact the stormwater staff of the governing agency about obtaining sanitary sewer permits within Ventura County. Discharges of certain types of flows to the sanitary sewer system may be cost prohibitive. The designer is urged to contact the appropriate agency prior to completing site and equipment design of the facility.

5.2 Description

Table 5-1 summarizes site-specific Source Control Measures and associated design features specified for various sites and activities. Fact Sheets are presented in this section for each source control measure. These sheets include design criteria established by the Approval Agencies to ensure effective implementation of the required Source Control Measures:

Table 5-1: Summary of Site-Specific Source Control Measure Design Features

Site-Specific Source Control Measure [1]	DESIGN FEATURE OR ELEMENT						
	Signs, placards, stencils	Surfacing (compatible, impervious)	Covers, screens	Grading/berming to prevent run-on	Grading/berming to provide secondary containment	Sanitary sewer connection	Emergency Storm Drain Seal
Storm Drain Message and Signage (S-1)	X						
Outdoor Material Storage Area Design (S-2)		X	X	X	X		X
Outdoor Trash Storage and Waste Handling Area Design (S-3)		X	X	X		X	
Outdoor Loading/Unloading Dock Area Design (S-4)		X	X	X	X		
Outdoor Repair/Maintenance Bay Design (S-5)		X	X	X	X		X
Outdoor Vehicle/Equipment/ Accessory Washing Area Design (S-6)		X	X	X	X	X	X
Fueling Area Design (S-7)		X	X	X	X		X
Parking Lot Design [2]							

1 Refer to Fact Sheets in Section 6 for detailed information and design criteria and Appendix E for BMP sizing worksheets

2 Requirements for proper design of parking lots are covered by requirements for General Site Design Principles and Techniques (see Section 4) and Treatment Control Measures (see Section 6).

5.3 Site-Specific Source Control Measures

S-1: Storm Drain Message and Signage

Purpose

Waste materials dumped into storm drain inlets can have severe impacts on receiving and ground waters. Posting notices regarding discharge prohibitions at storm drain inlets can prevent waste dumping. This Fact Sheet contains details on the installation of storm drain messages at storm drain inlets located in new or redeveloped commercial, industrial, and residential sites.

Design Criteria

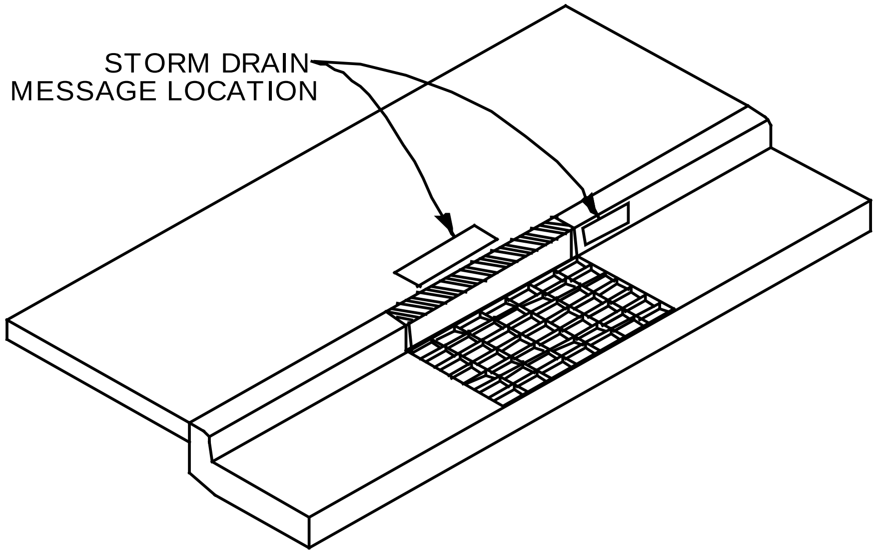
Storm drain messages have become a popular method of alerting the public to the effects of and the prohibitions against waste disposal into the storm drain system. The signs are typically stenciled or affixed near the storm drain inlet. The message simply informs the public that dumping of wastes into storm drain inlets is prohibited and/or the drain discharges to a receiving water.

Storm drain message markers or placards are required at all storm drain inlets within the boundary of the development project. The marker should be placed in clear sight facing anyone approaching the inlet from either side (see Figure 5-1). All storm drain inlet locations must be identified on the development site map.

Some local agencies within the County have approved storm drain message placards for use. Signs with language and/or graphical icons, which prohibit illegal dumping, should be posted at designated public access points along channels and streams within a project area. Consult local permitting agency stormwater staff to determine specific requirements for placard types and installation methods.

Maintenance Requirements

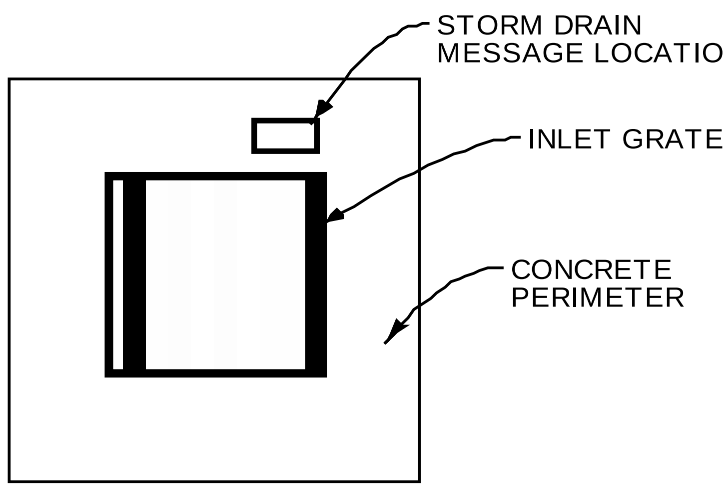
Legibility of markers and signs should be maintained. If required by the agency with jurisdiction over the project, the owner/operator or homeowner's association shall enter into a Maintenance Agreement with the agency or record a deed restriction upon the property title to maintain the legibility of placards and signs.



CURB TYPE INLET

NOTES:

1. **STORM DRAIN MESSAGE SHALL BE APPLIED IN SUCH A WAY AS TO PROVIDE A CLEAR, LEGIBLE IMAGE.**
2. **STORM DRAIN MESSAGE SHALL BE PERMANENTLY APPLIED DURING THE CONSTRUCTION OF THE CURB AND GUTTER USING A METHOD APPROVED BY THE LOCAL AGENCY.**



AREA TYPE INLET

Figure 5-1: Storm Drain Message Location

S-2: Outdoor Material Storage Area Design

Purpose

Materials that are stored outdoors could become sources of pollutants in stormwater runoff if not handled or stored properly. Materials could be in the form of raw products, by-products, finished products, and waste products. The type of pollutants associated with the materials will vary depending on the type of commercial or industrial activity.

Some materials are more of a concern than others. Toxic and hazardous materials must be prevented from coming in contact with stormwater. Non-toxic or non-hazardous materials do not have to be prevented from stormwater contact, but cannot be allowed to runoff with the stormwater. These materials may have toxic effects on receiving waters. Accumulated material on an impervious surface could result in significant debris and sediment being discharged with stormwater runoff causing a significant impact on the rivers or streams that receive the runoff.

Materials may be stored in a variety of ways, including bulk piles, containers, shelving, stacking, and tanks. Stormwater contamination may be prevented by eliminating the possibility of stormwater contact with the material storage areas either through diversion, cover, or capture of the stormwater. Control measures may also include minimizing the storage area. Control measures are site-specific and must meet local permitting agency requirements.

Design Criteria

Design requirements for material storage areas are governed by Building and Fire Codes and by current City or County ordinances and zoning requirements. Source Control Measures described in the Fact Sheet are intended to enhance and be consistent with these code and ordinance requirements. The following design features should be incorporated into the design of a material storage area when storing materials outside could contribute significant pollutants to the storm drain.

Table 5-2: Design Criteria for Outdoor Material Storage Area Design

Source Control Design Feature	Design Criteria
Surfacing	• Construct the storage area base with a material impervious to leaks and spills.
Covers	• Install a cover that extends beyond the storage area, or use a manufactured storage shed for small containers.
Grading/Containment	• Minimize the storage area. • Slope the storage area towards a dead-end sump to contain spills. • Grade or berm storage areas to prevent run-on from surrounding areas. • Direct runoff from downspouts/roofs away from storage areas.

Accumulated Stormwater and Non-stormwater

Stormwater and non-stormwater will accumulate in containment areas and sumps with impervious surfaces. Contaminated accumulated water must be disposed of in accordance with applicable laws and cannot be discharged directly to the storm drain or sanitary sewer system without the appropriate permit.

S-3: Outdoor Trash Storage Area Design

Purpose

Stormwater runoff from areas where trash is stored or disposed of can be polluted. In addition, loose trash and debris can be easily transported by water or wind into nearby storm drain inlets, channels, and/or creeks. Waste handling operations may be sources of stormwater pollution and include dumpsters, litter control, and waste piles. This fact sheet contains details on the specific measures required to prevent or reduce pollutants in stormwater runoff associated with trash storage and handling.

Design Criteria

Design requirements for waste handling areas are governed by Building and Fire Codes, and by current local permitting agency ordinances and zoning requirements. The design criteria described in the Fact Sheet are meant to enhance and be consistent with these code and ordinance requirements. Hazardous waste should be handled in accordance with legal requirements established in Title 22, California Code of Regulations.

Wastes from commercial and industrial sites are typically hauled by either public or commercial carriers that may have design or access requirements for waste storage areas. The design criteria listed below are recommendations and are not intended to be in conflict with requirements established by the waste hauler. The waste hauler should be contacted prior to the design of your site trash collection area to obtain established and accepted guidelines for designing trash collection areas. Conflicts or issues should be discussed with the local permitting agency.

The following trash storage area design controls were developed to enhance the local permitting agency codes and ordinances and should be implemented depending on the type of waste and the type of containment.

Table 5-3: Design Criteria for Outdoor Trash Storage Areas

Source Control Design Feature	Design Criteria
Surfacing	• Construct the storage area base with a material impervious to leaks and spills.
Screens/Covers	• Install a screen or wall around trash storage area to prevent offsite transport of loose trash. • Use lined bins or dumpsters to reduce leaking of liquid wastes. • Use water-proof lids on bins/dumpsters or provide a roof to cover enclosure (local permitting agency discretion) to prevent rainfall from entering containers.
Grading/Contouring	• Berm or grade the waste handling area to prevent run-on of stormwater. • Do not locate storm drains in immediate vicinity of the trash storage area.
Signs	• Post signs on all dumpsters informing users that hazardous materials are not to be disposed of therein.

Maintenance Requirements

The owner/operator must maintain the integrity of structural elements that are subject to damage (e.g. screens, covers and signs). Maintenance Agreements between the local permitting agency and the owner/operator may be required. Some agencies will require maintenance deed restrictions to be recorded of the property title. If required by the local permitting agency, Maintenance Agreements or deed restrictions must be executed by the owner/operator before improvement plans are approved. Refer to Appendix G and H for further guidance regarding Maintenance Plan Agreements.

S-4: Outdoor Loading/Unloading Dock Area Design

Purpose

Materials spilled, leaked, or lost during loading or unloading may collect on impervious surfaces or in the soil and be carried away by runoff or when the area is cleaned. Rainfall may also wash pollutants from machinery used to load or unload materials. Depressed loading docks (truck wells) are contained areas that can accumulate stormwater runoff. Discharge of spills or contaminated stormwater to the storm drain system is prohibited. This Fact Sheet contains details on specific measures recommended to prevent or reduce pollutants in stormwater runoff from outdoor loading or unloading areas.

Design Criteria

Design requirements for outdoor loading and unloading of materials are governed by Building and Fire Codes, and by current local permitting agency ordinances and zoning requirements. Source Control Measures described in this Fact Sheet are meant to enhance and be consistent with these code and ordinance requirements. Companies may have their own design or access requirements for loading docks. The design criteria listed below are not intended to be in conflict with requirements established by individual companies. Conflicts or issues should be discussed with the local permitting agency.

The following design criteria should be followed when developing construction plans for material loading and unloading areas:

Table 5-4: Design Criteria for Outdoor Loading/ Unloading Areas

Source Control Design Feature	Design Criteria
Surfacing	• Construct floor surfaces with materials that are compatible with materials being handled in the loading/unloading area.
Covers	• Cover loading/unloading areas to a distance of at least 3 feet beyond the loading dock or install a seal or door skirt to be used for all material transfers between the trailer and the building.
Grading/Contouring	• Grade or berm storage the areas to prevent run-on from surrounding areas. • Direct runoff from downspouts/roofs away from loading areas.
Emergency Storm Drain Seal	• Do not locate storm drains in the loading dock area. Direct connections to storm drains from depressed loading docks are prohibited. • Provide means, such as isolation valves, drain plugs, or drain covers, to prevent spills or contaminated stormwater from entering the storm drainage system.

Accumulated Stormwater and Non-stormwater

Stormwater and non-stormwater will accumulate in containment areas and sumps with impervious surfaces, such as depressed loading docks. Contaminated accumulated water must be disposed of in accordance with applicable laws and cannot be discharged directly to the storm drain or sanitary sewer system without the appropriate permit.

S-5: Outdoor Repair/Maintenance Bay Design

Purpose

Activities that can contaminate stormwater include engine repair, service, and parking (i.e. leaking engines or parts). Oil and grease, solvents, car battery acid, coolant and gasoline from the repair/maintenance bays can severely impact stormwater if allowed to come into contact with stormwater runoff. This Fact Sheet contains details on the specific measures required to prevent or reduce pollutants in stormwater runoff from vehicle and equipment maintenance and repair areas.

Design Criteria

Design requirements for vehicle maintenance and repair areas are governed by Building and Fire Codes, and by current local permitting agency ordinances, and zoning requirements. The design criteria described in this Fact Sheet are meant to enhance and be consistent with these code requirements.

The following design criteria are required for vehicle and equipment maintenance, and repair. All wash water, hazardous and toxic wastes must be prevented from entering the storm drainage system.

Source Control Design Feature	Design Criteria
Surfacing	• Construct the vehicle maintenance/repair floor area with Portland cement concrete.
Covers	• Cover or berm areas where vehicle parts with fluids are stored. • Cover or enclose all vehicle maintenance/repair areas.
Grading/ Contouring	• Berm or grade the maintenance/repair area to prevent run-on and runoff of stormwater or runoff of spills. • Direct runoff from downspouts/roofs away from maintenance/repair areas. • Grade the maintenance/repair area to drain to a dead-end sump for collection of all wash water, leaks and spills. Direct connection of maintenance/repair area to storm drain system is prohibited. • Do not locate storm drains in the immediate vicinity of the maintenance/repair area.
Emergency Storm Drain Seal	• Provide means, such as isolation valves, drain plugs, or drain covers, to prevent spills or contaminated stormwater from entering the storm drainage system.

Accumulated Stormwater and Non-stormwater

Stormwater and non-stormwater will accumulate in containment areas and sumps with impervious surfaces. Contaminated accumulated water must be disposed of in accordance with applicable laws and cannot be discharged directly to the storm drain or sanitary sewer system without the appropriate permit.

S-6: Outdoor Vehicle/Equipment/Accessory Washing Area Design

Purpose

Washing vehicles and equipment in areas where wash water flows onto the ground can pollute stormwater. Wash waters are not allowed in the storm drain system. They can contain high concentrations of oil and grease, solvents, phosphates and high suspended solids loads. Sources of washing contamination include outside vehicle/equipment cleaning or wash water discharge to the ground. This Fact Sheet contains details on the specific measures required to prevent or reduce pollutants in stormwater runoff from vehicle and equipment washing areas.

Design Criteria

Design requirements for vehicle maintenance and repair areas are governed by Building and Fire Codes, and by current local permitting agency ordinances, and zoning requirements. The design criteria described in this Fact Sheet are meant to enhance and be consistent with these code requirements.

The following design criteria are required for vehicle and equipment washing areas. All hazardous and toxic wastes must be prevented from entering the storm drain system.

Source Control Design Feature	Design Criteria
Surfacing	• Construct the vehicle/equipment wash area floors with Portland cement concrete.
Covers	• Provide a cover that extends over the entire wash area.
Grading/ Contouring	• Berm or grade the maintenance/repair area to prevent run-on and runoff of stormwater or runoff of spills. • Grade or berm the wash area to contain the wash water within the covered area and direct the wash water to treatment and recycle or pretreatment and proper connection to the sanitary sewer system. Obtain approval from the governing agency before discharging to the sanitary sewer. • Direct runoff from downspouts/roofs away from wash areas. • Do not locate storm drains in the immediate vicinity of the wash area.
Emergency Storm Drain Seal	• Provide means, such as isolation valves, drain plugs, or drain covers, to prevent spills or contaminated stormwater from entering the storm drainage system.

Accumulated Stormwater and Non-stormwater

Stormwater and non-stormwater will accumulate in containment areas and sumps with impervious surfaces. Contaminated accumulated water must be disposed of in accordance with applicable laws and cannot be discharged directly to the storm drain or sanitary sewer system without the appropriate permit.

S-7: Fueling Area Design

Purpose

Spills at vehicle and equipment fueling areas can be a significant source of pollution because fuels contain toxic materials and heavy metals that are not easily removed by stormwater treatment devices. When stormwater mixes with fuel spilled or leaked onto the ground, it becomes polluted by petroleum-based materials that are harmful to humans, fish, and wildlife. This could occur at large industrial sites or at small commercial sites such as gas stations and convenience stores. This Fact Sheet contains details on specific measures required to prevent or reduce pollutants in stormwater runoff from vehicle and equipment fueling areas, including retail gas stations.

Design Criteria

Design requirements for fueling areas are governed by Building and Fire Codes and by current local permitting agency ordinances and zoning requirements. The design requirements described in this Fact Sheet are meant to enhance and be consistent with these code and ordinance requirements.

Source Control Design Feature	Design Criteria
Surfacing	• Fuel dispensing areas must be paved with Portland cement concrete. The fuel dispensing area is defined as extending 6.5 feet from the corner of each fuel dispenser or the length at which the hose and nozzle assemble may be operated plus 1 foot, whichever is less. The paving around the fuel dispensing area may exceed the minimum dimensions of the "fuel dispensing area" stated above. • Use asphalt sealant to protect asphalt paved areas surrounding the fueling area.
Covers	• The fuel dispensing area must be covered [1], and the cover's minimum dimensions must be equal to or greater than the area within the grade break or the fuel dispensing area, as defined above. The cover must not drain onto the fuel dispensing area.
Grading/ Contouring	• The fuel dispensing area should have a 2% to 4% slope to prevent ponding and must be separated from the rest of the site by a grade break that prevents run-on of stormwater to the extent practicable. • Grade the fueling area to drain toward a dead-end sump. • Direct runoff from downspouts/roofs away from fueling areas. • Do not locate storm drains in the immediate vicinity of the fueling area.

Source Control Design Feature	Design Criteria
Emergency Storm Drain Seal	• Provide means, such as isolation valves, drain plugs, or drain covers, to prevent spills or contaminated stormwater from entering the storm drainage system.

1. If fueling large equipment or vehicles that would prohibit the use of covers or roofs, the fueling island should be designed to sufficiently accommodate the larger vehicles and equipment and to prevent run-on and runoff of stormwater. Grade to direct stormwater to a dead-end sump.

Accumulated Stormwater and Non-stormwater

Stormwater and non-stormwater will accumulate in containment areas and sumps with impervious surfaces. Contaminated accumulated water must be disposed of in accordance with applicable laws and cannot be discharged directly to the storm drain or sanitary sewer system without the appropriate permit.

S-8: Proof of Control Measure Maintenance

Purpose

Continued effectiveness of control measures specified in the 2011 TGM depends on diligent ongoing inspection and maintenance. To ensure that such maintenance is provided, the local permitting agency will require both a Maintenance Agreement and a Maintenance Plan from the owner/operator of stormwater control measures.

Maintenance Agreement

Onsite Treatment Control Measures are to be maintained by the owner/operator. Maintenance Agreements between the governing agency and the owner/operator may be required. A Maintenance Agreement with the governing agency must be executed by the owner/operator before occupancy of the project is approved. A sample Maintenance Agreement form is provided in Appendix H.

Maintenance Plan

A post-construction Maintenance Plan shall be prepared and made available at the governing agency's request. The Maintenance Plan should address items such as:

- Operation plan and schedule, including a site map;
- Maintenance and cleaning activities and schedule;
- Equipment and resource requirements necessary to operate and maintain facility; and
- Responsible party for operation and maintenance.

Additional guidelines for Maintenance Plans are provided in Appendix I.

6 STORMWATER BMP DESIGN

6.1 Introduction

Retention BMPs, Biofiltration BMPs, and Treatment Control Measures are required to augment Site Design Principles and Techniques and Source Control Measures to reduce pollution from stormwater discharges to the maximum extent practicable. Retention BMPs are engineered facilities that are designed to retain surface runoff on the project site. Biofiltration BMPs are vegetated stormwater BMPs that remove pollutants by filtering stormwater through vegetation and soils. Treatment Control Measures are engineered BMPs that provide a reduction of pollutant loads and concentrations in stormwater runoff. The type(s) of Retention BMPs and Biofiltration BMPs to be implemented depends on site suitability factors discussed in this chapter. The type of Treatment Control Measure(s) to be implemented at a site depends on a number of factors including: type of pollutants in the stormwater runoff, quantity of stormwater runoff to be treated, project site conditions, receiving water conditions, and state industrial permit requirements, where applicable. Land requirements and costs to design, construct, and maintain Treatment Control Measures vary by type.

Unlike flood control measures that are designed to handle peak flows, stormwater Retention BMPs, Biofiltration BMPs, and Treatment Control Measures are designed to retain or treat the more frequent, lower-flow storm events, or the first flush runoff from larger storm events (typically referred to as the first flush events). Small, frequent storm events represent most of the total average annual rainfall for the area. It's the volume from such small events, referred to as the Stormwater Quality Design Volume (SQDV), that is targeted for retention onsite in Retention BMPs. Biofiltration BMPs and Treatment Control Measures can be sized to capture either the SQDV or the Stormwater Quality Design Flow (SQDF). Calculation methods for the SQDV and the SQDF are presented in Section 2 and Appendix E.

6.2 General Considerations

Retention BMPs, Biofiltration BMPs, and Treatment Control Measures are designed to remove pollutants contained in stormwater runoff. The pollutants of concern, depending on the watershed, may include trash, debris, and sediment; metals such as copper, lead, and zinc; nutrients such as nitrogen and phosphorous; certain bacteria and viruses; mineral salts such as chloride; and organic chemicals such as petroleum hydrocarbons and pesticides. Pollutant removal methods include sedimentation/settling, filtration, plant uptake, ion exchange, adsorption, and microbially-mediated decomposition. Floatable pollutants such as oil, debris, and scum can be removed with separator structures. Retention BMPs, Biofiltration BMPs, and some Treatment Control Measures are also designed to reduce runoff volume, thereby reducing pollutant loading to receiving waters. Retention BMP, Biofiltration BMPs, and Treatment Control Measure types and common terms used in stormwater treatment are discussed below.

Maintenance Responsibility

Unless otherwise agreed to by the governing stormwater agency, the landowner, site operator, or homeowner's association is responsible for the operation and maintenance of the Retention BMPs, Biofiltration BMPs, and Treatment Control Measures. Failure to properly operate and maintain the measures could result in reduced treatment of stormwater runoff or a concentrated loading of pollutants to the storm drain system. To protect against failure, a Maintenance Plan must be developed and implemented for all Retention BMPs, Biofiltration BMPs, and Treatment Control Measures. Guidelines for maintenance plans are provided in Appendix I of the 2011 TGM. The Plan must be made available at the agency's request. In addition, a maintenance agreement with the governing agency may be required. The example maintenance agreements are included in Appendix H.

In addition to maintenance, the governing agency may require water quality monitoring agreements for any of the Retention BMPs, Biofiltration BMPs, or Treatment Control Measures recommended in the 2011 TGM. Monitoring may be conducted by the site operator, the agency, or both. Monitoring may be required for a period of time to help the agency evaluate the effectiveness of Retention BMPs, Biofiltration BMPs, and Treatment Control Measures in reducing pollutants in stormwater runoff.

Pretreatment

Pretreatment must be provided for filtration and infiltration facilities and other facilities whose function could be adversely affected by sediment or other pollutants. Pretreatment may also be provided for water quality detention basins and other Treatment Control Measures to facilitate the routine removal of sediment, trash, and debris, and to increase the longevity of the downstream BMPs.

Pretreatment may be provided by presettling basins or forebays (small detention basins), vegetated swales, filter strips, and hydrodynamic separators. Source control activities, described in Chapter 5, minimize the introduction of pollutants into stormwater runoff and also help to protect filtration and infiltration facilities. Effort should be made early in the site planning stages to minimize runoff from impervious areas by grading toward landscaped areas, disconnecting downspouts, and using pervious conveyances prior to discharging to the storm drain system. These site design practices can reduce the size and maintenance burden of downstream, end-of-pipe BMPs.

Oil/Water Separation

Oil/water separators remove floating oil from the water surface. There are two general types of separators: American Petroleum Institute (API) separators and coalescing plate (CP) separators. Both types use physical mechanisms to remove high concentrations of floating and dispersed oil. Oil/water separators are not suitable for the relatively low concentrations of petroleum hydrocarbons present in typical urban runoff, and should only be used in locations where higher concentrations of oil are expected to occur, such as retail fuel facilities, high volume roads, and petroleum-related industrial facilities. Oil/water separators must be located off-line from the primary conveyance system, as they function at low flow conditions and will wash out in high flow conditions. Other oil control devices/facilities that may be used for pretreatment of slightly elevated concentrations of oil (i.e., typical of high use commercial parking lots) include catch basin inserts, hydrodynamic devices, and linear sand filters. Oil control devices/facilities should always be placed upstream of other treatment facilities and as close to the oil source as possible.

Infiltration

Infiltration refers to the use of the filtration, adsorption, and biological decomposition properties of soils to remove pollutants prior to the intentional routing of runoff to the subsurface for groundwater recharge. Infiltration BMPs are a type of Retention BMP and include infiltration basins, infiltration trenches, bioretention without an underdrain, dry wells, permeable pavement, and proprietary infiltration devices. Infiltration can provide multiple benefits including pollutant removal, hydromodification control, groundwater recharge, and flood control. However, conditions that can limit the use of infiltration include soil properties and potential adverse impacts on groundwater quality. A geotechnical investigation must be conducted when evaluating infiltration to determine the suitability of the site soil in adequately addressing groundwater protection. This may include an in-situ percolation test, per the guidance provided in Appendix C, and the determination of minimum depth to groundwater. The minimum separation to seasonal high groundwater or estimated mounded groundwater is five feet. Depth to seasonal high groundwater level shall be estimated as the average of the annual minima (i.e., the shallowest recorded measurements in each water year, defined as October 1 through September 30) for all years on record. If groundwater level data are not available or not considered to be representative, seasonal high groundwater depth can be determined by redoximorphic analytical methods combined with temporary groundwater monitoring for November 1 through April 1 at the proposed project site.

Soils should have sufficient organic content and sorption capacity to remove certain pollutants, but must be coarse enough to infiltrate runoff in a reasonable amount of time (e.g., < 72 hours for above-ground ponded water to prevent vector breeding). Examples of suitable soils are silty and sandy loams. Coarser soils, such as gravelly sands, have limited organic content and high permeability and therefore present a potential risk to groundwater from certain pollutants, especially in areas of shallow groundwater. Prior to the use of infiltration BMPs, consult with the local permitting agency to identify if vulnerable unconfined aquifers are located beneath the project to determine the appropriateness of these BMPs. In an area identified as an unconfined aquifer, the application of infiltration BMPs should include significant pretreatment to ensure groundwater is protected from pollutants of concern.

Infiltration BMPs should not be placed in high-risk areas such as at or near service/gas stations, truck stops, and heavy industrial sites due to the groundwater contamination risk. Infiltration BMPs may be placed in high-risk areas if a site-specific evaluation demonstrates that sufficient pretreatment is provided to address pollutants of concern, high risks areas are isolated from stormwater runoff, or infiltration areas have little chance of spill migration.

In addition, infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project. Adequate spacing (100 feet or more) must be provided between infiltration BMPs and potable wells, non-potable wells, drain fields, and springs. Infiltration BMPs must be setback from building foundations at least eight feet or have an alternative setback established by the geotechnical expert for the project.

Infiltration is not allowed at locations with contaminated soils or groundwater where the pollutants could be mobilized or exacerbated by infiltration, unless a site-specific analysis determines the infiltration would not be detrimental. A site-specific analysis shall be prepared where pollutant mobilization (e.g., naturally-derived groundwater pollutants) is a concern. Projects must consider the potential for mobilization of groundwater contamination from natural sources as a result of stormwater infiltration (e.g., marine sediments, selenium-rich groundwater) to the extent that data is available.

Incidental infiltration that occurs in other types of Biofiltration BMPs and Treatment Control Measures, such as dry extended detention basins, vegetation swales, filter strips, and bioretention areas with underdrains, pose little risk to groundwater quality as treatment is provided in the BMP prior to infiltration.

Biofiltration BMPs

Biofiltration BMPs use vegetation and soils or other filtration media for runoff treatment. As runoff passes through the vegetation and filtration media, the combined effects of filtration, adsorption, and biological uptake remove pollutants. In biofiltration BMPs, pore spaces and organic material in the soils help to retain water in the form of soil moisture and to promote the pollutant adsorption (e.g., dissolved metals and petroleum hydrocarbons) into the soil matrix. Plants use soil moisture, promote the drying of the soil through transpiration, and uptake pollutants in their roots and leaves. Plants with extensive root systems also help to maintain filtration rates. Vegetation also decreases the velocity of flow and allows for particulates to settle.

Treatment Control Measures

Filtration

Various media, such as sand, perlite, zeolite, compost, and activated carbon, can be used in filtration BMPs to effectively remove total suspended solids (TSS) and associated pollutants such as organics (hydrocarbons and pesticides) and particulate metals. Filtration systems can be configured in the form of horizontal beds, trenches, or lastly, cartridge systems in underground vaults or catch basins.

Wetpools

A wetpool is a permanent pool of water incorporated into a wetpond or stormwater wetland BMP. Wetpools provide runoff treatment by allowing settling of particulates (sedimentation) by biological uptake and by vegetative filtration (if vegetation is present). Wetpool BMPs may be single-purpose facilities, providing only runoff treatment, or they may also provide flow control by providing additional detention storage with the use of a multi-stage outlet structure. If combined with detention, the wetpool volume can often be stacked under the detention volume with little further loss of development area.

"On-line" and "Off-line" Facilities

The location and configuration of control facilities can vary depending on the desired function. For example, drop structures or grade control may be located in a drainage channel so as to stabilize a channel for hydromodification control purposes. Such facilities are referred to as "in-stream" controls. Retention BMPs, Biofiltration BMPs, and Treatment Control Measures may not be located in-stream. Retention BMPs, Biofiltration BMPs, and Treatment Control Measures cannot be located in Waters of the US, but rather must be located upland to retain or treat runoff prior to discharge into Waters of the US.

If a Retention BMP, Biofiltration BMP, or Treatment Control Measure facility is designed such that all the runoff passes through the facility, the facility is called an "on-line" system. However, care must be taken to limit the resuspension of previously captured pollutants or damage to BMP performance during high flows. If, on the other hand, the facility only receives flows less than or equal to the stormwater quality design flow (SQDF), the facility is called an "off-line" system. Off-line systems therefore require a flow splitter or equivalent device. Generally treatment performance is better for off-line facilities because a larger percentage of the runoff is treated. Figure 6-1 illustrates the difference between on-line, off-line, and in-stream controls.

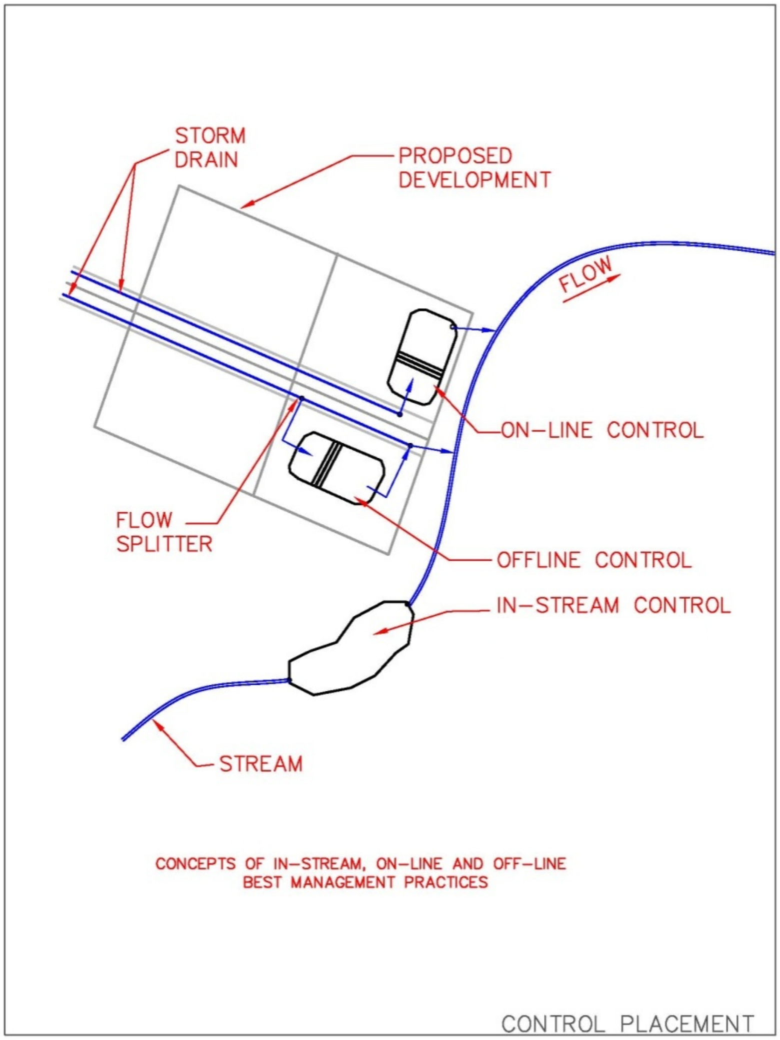


Figure 6-1: Differences between On-line, Off-line, and In-stream Control Measures

6.3 Retention BMP, Biofiltration BMP, and Treatment Control Measure Fact Sheets

This section provides fact sheets with recommended criteria for the design and implementation of Retention BMPs, Biofiltration BMPs, and Treatment Control Measures. The siting, design, and maintenance requirements in the fact sheets are intended to ensure optimal performance of the measures. Alternative designs may be approved by the local permitting authority based on site specific conditions if equivalent pollutant removal performance is provided.

The 2011 TGM also contains calculation worksheets to aid in the design of these BMPs in Appendix E. New BMPs that are equivalent to those included in the 2011 TGM are acceptable based on approval of the local permitting agency.

Fact sheets are provided for the Retention BMPs, Biofiltration BMPs, and Treatment Control Measures listed below:

Retention BMPs

Infiltration BMPs

INF-1: Infiltration Basin
INF-2: Infiltration Trench
INF-3: Bioretention
INF-4: Drywell
INF-5: Permeable Pavement
INF-6: Proprietary Infiltration

Rainwater Harvesting BMPs

RWH-1: Rainwater Harvesting

Evapotranspiration BMPs

ET-1: Green Roof
ET-2: Hydrologic Source Controls

Biofiltration BMPs

BIO-1: Bioretention with Underdrain
BIO-2: Planter Box
BIO-3: Vegetated Swale
BIO-4: Vegetated Filter Strip
BIO-5: Proprietary Biotreatment

Treatment Control Measures

TCM-1: Dry Extended Detention Basin
TCM-2: Wet Detention Basin
TCM-3: Constructed Wetland
TCM-4: Sand Filter (if vegetated, this is considered a Biofiltration BMP)
TCM-5: Cartridge Media Filter

Pretreatment/Gross Solids Removal BMPs

PT-1: Hydrodynamic Device
PT-2: Catch Basin Insert

INF-1: Infiltration Basin

An infiltration basin consists of an earthen basin constructed in naturally pervious soils (Type A or B soils) with a flat bottom and provided with an inlet structure to dissipate energy of incoming flow and an emergency spillway to control excess flows. An optional relief underdrain may be provided to drain the basin if standing water conditions occur. A forebay settling basin or separate Treatment Control Measure must be provided as pretreatment. An infiltration basin functions by retaining the SQDV in the basin and allowing the retained runoff to percolate into the underlying native soils over a specified period of time. The bottoms of infiltration basins are typically vegetated with dry-land grasses or irrigated turf grass. A typical layout of an infiltration basin system is shown in Figure 6-2.

Infiltration Basin in a Fresno, CA Park, Before and After a Rain Event

Photo Credit: Geosyntec Consultants

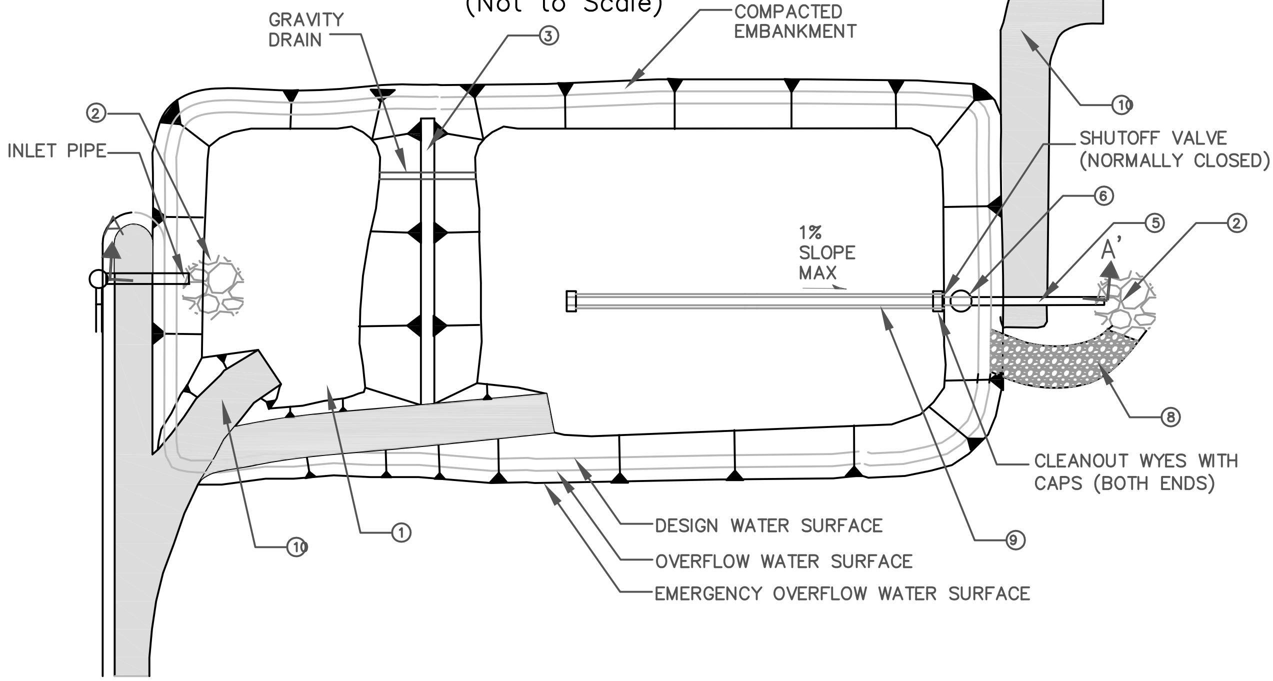
Application

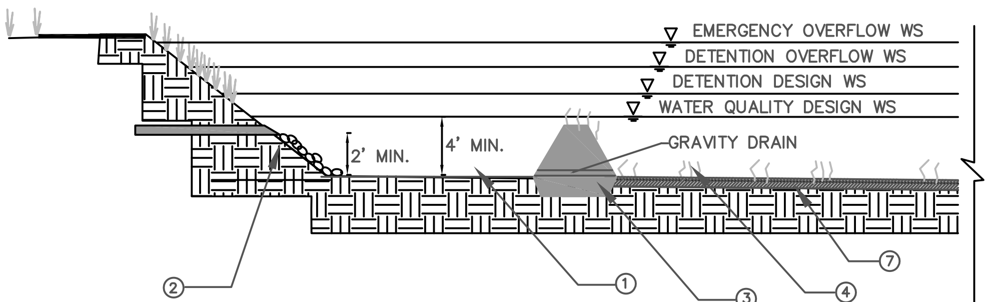
- Mixed-use and commercial
- Roads and parking lots
- Parks and open spaces
- Single and multi-family residential
- Can integrate with parks

Routine Maintenance

- Removal trash, debris, and sediment at inlet and outlets
- Wet weather inspection to ensure drain time
- Remove weeds
- Inspect for mosquito breeding

Plan View
(Not to Scale)





Section A – A'
(Not to Scale)

NOTES:

1. UPSTREAM PRETREATMENT SHALL BE PROVIDED. SEDIMENT FOREBAY WITH VOLUME EQUAL TO 25% OF TOTAL INFILTRATION BASIN VOLUME MAY BE USED IN LIEU OF UPSTREAM PRETREATMENT. DEPTH SHALL BE 4' MIN TO 8' MAX PLUS AN ADDITIONAL 1 FOOT MIN SEDIMENT STORAGE DEPTH.
2. RIP RAP APRON OR OTHER ENERGY DISSIPATION.
3. EXTEND EARTHEN BERM ACROSS ENTIRE WIDTH OF THE INFILTRATION BASIN.
4. INFILTRATION BASIN BOTTOM AND SIDE SLOPES SHALL BE PLANTED WITH DROUGHT TOLERANT VEGETATION. DEEP ROOTED VEGETATION PREFERRED FOR BASIN BOTTOM. NO TOPSOIL SHALL BE ADDED TO INFILTRATION BASIN BED.
5. SIZE OUTLET PIPE TO PASS 100–YEAR PEAK FLOW FOR ON–LINE INFILTRATION BASINS AND WATER QUALITY PEAK FLOW FOR OFF–LINE INFILTRATION BASINS.
6. WATER QUALITY OUTLET STRUCTURE. SEE FIGURE 7–2 AND FIGURE 7–3 FOR DETAILS.
7. OVER EXCAVATE BASIN BOTTOM 1 FOOT. RE–PLACE EXCAVATED MATERIAL UNIFORMLY WITHOUT COMPACTION. AMENDING EXCAVATED MATERIAL WITH 2" – 4" OF COARSE SAND IS RECOMMENDED FOR SOILS WITH BORDER LINE INFILTRATION CAPACITY.
8. INSTALL EMERGENCY OVERFLOW SPILLWAY AS NEEDED. SEE FIGURE 2–4 FOR DETAILS
9. INSTALL OPTIONAL 6" MINIMUM DIAMETER PERFORATED PIPE UNDERDRAIN. INSTALL AT 0.5% MINIMUM SLOPE.
10. MAINTENANCE RAMP SHOULD PROVIDE ACCESS TO BOTH THE FIRST CELL AND MAIN BASIN.

Geosyntec consultants

Figure 6-2: Infiltration Basin

Limitations

The following limitations should be considered before choosing to use an infiltration basin:

- Native soil infiltration rate - permeability of soils at the infiltration basin location must be at least 0.5 inches per hour.
- Depth to groundwater, bedrock, or low permeability soil layer – 5 feet vertical separation is required between the bottom of the infiltration basin and the seasonal high groundwater level or mounded groundwater level, bedrock, or other barrier to infiltration to ensure that the facility will completely drain between storms and that infiltrating water will receive adequate treatment though the soils before it reaches the groundwater.
- Slope stability - infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.
- Setbacks - a minimum setback (100 feet or more) must be provided between infiltration BMPs and potable wells, non-potable wells, drain fields, and springs. Infiltration BMPs must be setback at least eight feet from building foundations or have an alternative setback established by the geotechnical expert for the project.
- Groundwater contamination - the application of infiltration BMPs should include significant pretreatment in an area identified as an unconfined aquifer to ensure groundwater is protected for pollutants of concern.
- Contaminated soils or groundwater plumes - infiltration BMPs are not allowed at locations with contaminated soils or groundwater, where the pollutants could be mobilized or exacerbated by infiltration, unless a site-specific analysis determines the infiltration would be beneficial.
- High pollutant land uses - infiltration BMPs should not be placed in high-risk areas such as at or near service/gas stations, truck stops, and heavy industrial sites due to the groundwater contamination risk unless a site-specific evaluation demonstrates that sufficient pretreatment is provided to address pollutants of concern, high risks areas are isolated from stormwater runoff, or infiltration areas have little chance of spill migration.
- High sediment loading rates – infiltration BMPs may clog quickly if sediment loads are high (e.g., unstabilized site) or if flows are not adequately pretreated.

Additional Control Functions

Infiltration basins can be designed for flow control by providing storage capacity in excess of that provided by infiltration and incorporating outlet controls. The additional storage and outlet structure should be provided per the requirements outlined in the Dry Extended Detention Basins section of the 2011 TGM. Note that the selected outlet structure should not be designed to drain the design volume intended for infiltration and should be similar to outlet structures that maintain a permanent pool (see Section 6.10.2 – Wet Retention Basins).

Multi-Use Opportunities

Infiltration basins may be integrated into the design of a park or playfield. Recreational multi-use facilities should be inspected after every storm and may require a greater maintenance frequency than dedicated infiltration basins to ensure aesthetics and public safety are not compromised. Any planned multi-use facility must obtain approval by the affected City and County departments.

Design Criteria

The main challenge associated with infiltration basins is preventing system clogging and subsequent infiltration inhibition. Infiltration basins should be designed according to the requirements listed in Table 6-1 and outlined in the section below. Detailed design procedures and an example are included in Appendix E.

Table 6-1: Infiltration Basin Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume (SQDV)	acre-feet	See Section 2.3 and Appendix E for calculating SQDV
Design drawdown time	hr	12 - 72 (See Appendix D, Section D.2)
Bottom basin Elevation	feet	5 feet above seasonally high groundwater table or mounded groundwater
Setbacks	feet	100 feet from wells, fields, and springs; 20 feet downslope of 100 feet upslope of foundations; Geotechnical expert should establish the setback requirement from building foundations that must be ≥ 8 ft.
Pretreatment	-	Sedimentation forebay or any Treatment Control Measure shall be provided as pretreatment for all tributary surfaces other than roofs.

Design Parameter	Unit	Design Criteria
Design percolation rate (P_{design})	in/hr	Measured percolation rate must be corrected based onsite suitability assessment and design related considerations described in this fact sheet.
Facility geometry	-	Forebay (if applicable): 25% of facility volume; flat bottom slope
Freeboard (minimum)	ft	1.0
Inlet/ Outlet erosion control	-	Energy dissipater to reduce velocity
Overflow device	-	Required if system is on-line

Geotechnical Considerations

An extensive geotechnical site investigation must be undertaken early in the site planning process to verify site suitability for the installation of infiltration facilities, due to the potential to contaminate groundwater, cause slope instability, impact surrounding structures, and have insufficient infiltration capacity.. Soil infiltration rates and the water table depth should be evaluated to ensure that conditions are satisfactory for proper operation of an infiltration facility. See Appendix C for guidance on infiltration testing.

The project designer must demonstrate through infiltration testing, soil logs, and the written opinion of a licensed civil engineer that sufficiently permeable soils exist onsite to allow the construction of a properly functioning infiltration facility.

1) Infiltration facilities require a minimum soil infiltration rate of 0.5 inches/hour. Pretreatment is required in all instances.

2) Groundwater separation must be at least 5 feet from the basin bottom to the measured Seasonal High Groundwater Elevation or estimated high groundwater mounding elevation. Groundwater levels measurements must be made during the time when water level is expected to be at a maximum (i.e., toward the end of the wet season).

3) Potential BMP sites with a slope greater than 25% (4:1) should be excluded. A geotechnical analysis and report addressing slope stability are required if located within 50 feet of slopes greater than 15%.

Soil Assessment and Site Geotechnical Investigation Reports

The soil assessment report should:

- State whether the site is suitable for the proposed infiltration basin;

- Recommend a design percolation rate (see "*Step 2: Determine The Design Percolation Rate*" below);
- Identify the seasonally high depth to groundwater table surface elevation;
- Provide a good understanding of how the stormwater runoff will move in the soil (horizontally or vertically) and if there are any geological conditions that could inhibit the movement of water; and
- If a geotechnical investigation and report are required, the report should:
  - Provide a written opinion by a professional civil engineer describing whether the infiltration basin will compromise slope stability; and
  - Identify potential impacts to nearby structural foundations.

Setbacks

1) Infiltration facilities shall be setback a minimum of 100 feet from proposed or existing potable wells, non-potable wells, septic drain fields, and springs.
2) Infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.
3) The geotechnical expert shall establish the setback requirement from building foundations that must be ≥ 8 ft.

Pretreatment

Pretreatment is required for infiltration basins in order to reduce the sediment load entering the facility and maintain the infiltration rate of the facility. Pretreatment refers to design features that provide settling of large particles before runoff reaches a management practice; easing the long-term maintenance burden. Pretreatment is important for most all structural stormwater BMPs, but it is particularly important for infiltration BMPs. To ensure that pretreatment mechanisms are effective, designers should incorporate sediment reduction practices. Sediment reduction BMPs may include vegetated swales, vegetated filter strips, sedimentation basins or forebays, sedimentation manholes and hydrodynamic separation devices. The use of at least two pretreatment devices is highly recommended for infiltration basins.

For design specification of selected pretreatment devices, refer to:

- BIO-3: Vegetated swales
- BIO-4: Vegetated filter strips
- TCM-4: Sand filters

- TCM-5: Cartridge media filters
- PT-1: Hydrodynamic separation device

Sizing Criteria

As with sand filters, infiltration facilities can be sized using one of two methods: a simple sizing method or a routing modeling method. With either method the SQDV volume must be completely infiltrated within 12 to 72 hours (see Appendix D, Section D.2 for a discussion on drawdown time and BMP performance). The simple sizing procedures provided below can be used for either infiltration basins or infiltration trenches (see INF-2: Infiltration Trench). For the routing modeling method, refer to TCM-4 Sand Filters.

Step 1: Calculate the Design Volume

Infiltration facilities shall be sized to capture and infiltrate the SQDV volume (see Section 2 and Appendix E) with a 12 to 72 hour drawdown time (see Appendix D, Section D.2).

Step 2: Determine the Design Percolation Rate

The percolation rate will decline between maintenance cycles as the surface becomes occluded and particulates accumulate in the infiltrative layer. Monitoring of actual facility performance has shown that the full-scale infiltration rate is far lower than the rate measured by small-scale testing. It is important that adequate conservatism is incorporated in the selection of design percolation rates. For infiltration trenches, the design percolation rate discussed here is the percolation rate of the underlying soils and not the percolation rate of the filter media bed (refer to the "Geometry and Sizing" section of INF-2 for the recommended composition of the filter media bed for infiltration trenches).

<u>Considerations for Design Percolation Rate Corrections</u>

Suitability assessment related considerations include (Table 6-2):

- Soil assessment methods – the site assessment extent (e.g., number of borings, test pits, etc.) and the measurement method used to estimate the short-term infiltration rate.
- Predominant soil texture/percent fines – soil texture and the percent fines can greatly influence the potential for clogging.
- Site soil variability – site with spatially heterogeneous soils (vertically or horizontally), as determined from site investigations, are more difficult to estimate average properties resulting in a higher level of uncertainty associated with initial estimates.

- Depth to seasonal high groundwater/impervious layer – groundwater mounding may become an issue during excessively wet conditions where shallow aquifers or shallow clay lenses are present.

Table 6-2: Suitability Assessment Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern	Medium Concern	Low Concern
Assessment methods	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates	Direct measurement of ≥ 20 percent of infiltration area with localized infiltration measurement methods (e.g., infiltrometer)	Direct measurement of ≥ 50 percent of infiltration area with localized infiltration measurement methods or Use of extensive test pit infiltration measurement methods
Ventura Hydrology Manual soil number (measured infiltration rate)	3 (f = 0.5 – 0.64)	4 or 5 (f = 0.65 –0.91)	6 or 7 (f = 0.92 or higher)
Site soil variability	Highly variable soils indicated from site assessment or limited soil borings collected during site assessment	Soil borings/test pits indicate moderately homogeneous soils	Multiple soil borings/test pits indicate relatively homogeneous soils
Depth to groundwater/ impervious layer	<10 ft below facility bottom	10-30 ft below facility bottom	>30 below facility bottom

Localized infiltration testing refers to methods such as the double ring infiltrometer test (ASTM D3385-88), which measure infiltration rates over an area less than 10 sq-ft and do not attempt to account for soil heterogeneity. Extensive infiltration testing refers to methods that include excavating a significant portion of the proposed infiltration area, filling the excavation with water, and monitoring drawdown. In all cases, testing should be conducted in the area of the proposed BMP where, based on geotechnical data, soils appear least likely to support infiltration.

Design related considerations include (Table 6-3):

- Size of area tributary to facility – all things being equal, both physical and economic risk factors related to infiltration facilities increase with an increase in the tributary area served. Therefore facilities serving larger tributary areas should use more restrictive adjustment factors.

- Level of pretreatment/expected influent sediment loads – credit should be given for good pretreatment by allowing less restrictive factors to account for the reduced probability of clogging from high sediment loading. Also, facilities designed to capture runoff from relatively clean surfaces such as rooftops are likely to see low sediment loads and therefore should be allowed to apply less restrictive safety factors.

- Redundancy – facilities that consist of multiple subsystems operating in parallel such that parts of the system remains functional when other parts fail and/or bypass, should be rewarded for the built-in redundancy with less restrictive correction and safety factors. For example, if bypass flows would be at least partially treated by another BMP, the risk of discharging untreated runoff in the event of clogging the primary facility is reduced. A bioretention facility that overflows to a landscaped area is another example. Compaction during construction – proper construction oversight is needed during construction to ensure that the bottoms of infiltration facility are not overly compacted. Facilities that do not commit to proper construction practices and oversight should have to use more restrictive correction and safety factors.

Table 6-3: Design Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern	Medium Concern	Low Concern
Tributary area size	Greater than 10 acres.	Greater than 2 acres but less than 10 acres.	2 acres or less.
Level of pre-treatment/ expected influent sediment loads	Pre-treatment from gross solids removal devices only, such as hydrodynamic separators, racks and screens, AND tributary area includes landscaped areas, steep slopes, high traffic areas, or any other areas expected to produce high sediment, trash, or debris loads.	Good pre-treatment with BMPs that mitigate coarse sediments such as vegetated swales AND influent sediment loads from the tributary area are expected to be relatively low (e.g., low traffic, mild slopes, disconnected impervious areas, etc.).	Excellent pre-treatment with BMPs that mitigate fine sediments such as bioretention or media filtration OR sedimentation or facility only treats runoff from relatively clean surfaces, such as rooftops.
Redundancy of treatment	No redundancy in BMP treatment train.	Medium redundancy, other BMPs available in treatment train to maintain at least 50% of function of facility in event of failure.	High redundancy, multiple components capable of operating independently and in parallel, maintaining at least 90% of facility functionality in event of failure.
Compaction during construction	Construction of facility on a compacted site or elevated probability of unintended/ indirect compaction.	Medium probability of unintended/ indirect compaction.	Heavy equipment actively prohibited from infiltration areas during construction and low probability of unintended/ indirect compaction.

Adjust the measured short-term infiltration rate using a weighted average of several safety factors using the worksheet shown in Table 6-4 below. The design percolation rate would be determined as follows:

- For each consideration shown in Table 6-2 and Table 6-3 above, determine whether the consideration is a high, medium, or low concern.
- For all high concerns, assign a factor value of 3, for medium concerns, assign a factor value of 2, and for low concerns assign a factor value of 1.
- Multiply each of the factors by the corresponding weight to get a product.

- Sum the products within each factor category to obtain a safety factor for each.
- Multiply the two safety factors together to get the final combined safety factor. If the combined safety factor is less than 2, then use 2 as the safety factor.
- Divide the measured short-term infiltration rate by the combined safety factor to obtain the adjusted design percolation rate for use in sizing the infiltration facility.

Table 6-4: Infiltration Facility Safety Factor Determination Worksheet

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) p = w x v
A	Suitability Assessment	Soil assessment methods	0.25		
		Predominant soil texture	0.25		
		Site soil variability	0.25		
		Depth to groundwater / impervious layer	0.25		
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			
B	Design	Tributary area size	0.25		
		Level of pre-treatment/ expected sediment loads	0.25		
		Redundancy	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \Sigma p$			
Combined Safety Factor = S_A x S_B					

Note: The minimum combined adjustment factor shall not be less than 2.0 and the maximum combined adjustment factor shall not exceed 9.

Step 3: Calculate the surface area

Determine the size of the required infiltrating surface by assuming the SQDV will fill the available ponding depth plus (for infiltration trenches) the void spaces based on the computed porosity of the filter media (normally about 32%).

1) Determine the maximum depth of runoff that can be infiltrated within the required drain time (d_{max}) as follows:

$$d_{\max} = \frac{P_{design}}{12} t \qquad \text{(Equation 6-1)}$$

Where:

d_{max}	=	the maximum depth of water that can be infiltrated within the required drain time (ft)
P_{design}	=	design percolation rate of underlying soils (in/hr)
t	=	required drain time (hrs)

2) Choose the ponding depth (d_p) and/or trench depth (d_t) such that:

$$d_{\max} \geq d_p$$ For Infiltration Basins (Equation 6-2)

$$d_{\max} \geq n_t d_t + d_p$$ For Infiltration Trenches (Equation 6-3)

Where:

d_{max}	=	the maximum depth of water that can be infiltrated within the required drain time (ft)
d_p	=	ponding depth (ft)
n_t	=	trench fill aggregate porosity (unitless)
d_t	=	depth of trench fill (ft)

3) Calculate infiltrating surface area (filter bottom area) required:

$$A = \frac{SQDV}{((TP_{design}/12) + d_p)}$$ For Infiltration Basins (Equation 6-4)

$$A = \frac{SQDV}{((TP_{design}/12) + n_t d_t + d_p)}$$ For Infiltration Trenches (Equation 6-5)

Where:

$SQDV$	=	stormwater quality design volume (ft^3)
n_t	=	trench fill aggregate porosity (unitless)
P_{design}	=	design percolation rate (in/hr)
d_p	=	ponding depth (ft)
d_t	=	depth of trench fill (ft)
T	=	fill time (time to fill to max ponding depth with water) (hrs) [use 2 hours for most designs]

Geometry and Sizing

1) Infiltration basins should be designed and constructed with the flattest bottom slope possible to promote uniform ponding and infiltration across the facility.

2) A sediment forebay is required unless adequate pretreatment is provided in a separate pretreatment unit (e.g., vegetated swale, filter strip, hydrodynamic device) to reduce sediment loads entering the infiltration basin. The sediment forebay, if present, should have a volume equal to 25% of the total infiltration basin volume.

3) The forebay should be designed with a minimum length to width ratio of 2:1 and should completely drain to the main basin through an 8-inch minimum low-flow outlet within 10 minutes.

4) All inlets should enter the sediment forebay. If there are multiple inlets, the length-to-width ratio should be based on the average flowpath length for all inlets.

5) Design embankments to conform to requirements of the State of California Division of Safety of Dams, if the basin dimensions cause it to fall under that agency's jurisdiction.

Drainage

1) The bottom of the infiltration bed should be native soil, over-excavated to at least one foot in depth, and replaced uniformly without compaction. Amending the excavated soil with 2-4 inches (~15-30%) of coarse sand is recommended.

2) The hydraulic conductivity of the subsurface layers should be sufficient to ensure a maximum 72-hr drawdown time. An observation well shall be incorporated to allow observation of drain time.

3) For infiltration basins, an underdrain should be installed within the bottom layer to provide drainage in case of standing water. The underdrain should be operated by opening a valve, which should be closed during normal operation. Cleanouts should be provided for the underdrain. See Sand Filter Section VEG-8 for specifications for underdrains.

Emergency Overflow

1) There should be an overflow route for stormwater flows that overtop the facility or in case the infiltration facility becomes clogged.

2) The overflow channel should be able to safely convey flows from the peak design storm to the downstream stormwater conveyance system or other acceptable discharge point.

3) Spillway and overflow structures should be designed in accordance with applicable standards of the Ventura County Flood Control District or local jurisdiction.

Vegetation

1) A thick mat of drought tolerant grass should be established on the basin floor and side-slopes following construction. Grasses can help prevent erosion and increase evapotranspiration and their roots discourage compaction helping to maintain the surface infiltration rates. Additionally, the active growing vegetation can help break up surface layers that accumulate fine particulates.

2) Grass may need to be irrigated during establishment.

3) For infiltration basins, landscaping of the area surrounding the basin should adhere to the following criteria so as not to hinder maintenance operations:

    a. No trees or shrubs may be planted within 10 feet of inlet or outlet pipes or manmade drainage structures such as spillways, flow spreaders, or earthen embankments. Species with roots that seek water, such as willow or poplar, should not be used within 50 feet of pipes.

    b. Prohibited non-native plant species will not be permitted. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website or the California Invasive Plant Council website at www.cal-ipc.org.

Maintenance Access

1) Maintenance access road(s) shall be provided to the drainage structures associated with the basin (e.g., inlet, emergency overflow, or bypass structures). Manhole and catch basin lids should be in or at the edge of the access road.

2) An access ramp to the basin bottom is required to facilitate the entry of sediment removal and vegetation maintenance equipment without compaction of the basin bottom and side slopes.

Construction Considerations

To preserve and avoid the loss of infiltration capacity, the following construction guidelines are specified:

1) The entire area draining to the facility should be stabilized before construction begins. If this is impossible, a diversion berm should be placed around the perimeter of the infiltration site to prevent sediment entrance during construction.

2) Infiltration basins should not be hydraulically connected to the stormwater conveyance system until all contributing tributary areas are stabilized as shown on the Contract Plans and to the satisfaction of the Engineer. Infiltration basins should not be used as sediment control facilities.

3) Compaction of the subgrade with heavy equipment should be minimized to the maximum extent possible. If the use of heavy equipment on the base of the facility cannot be avoided, the infiltrative capacity should be restored by tilling or aerating prior to placing the infiltrative bed.

4) The exposed soils should be inspected by a civil engineer after excavation to confirm that soil conditions are suitable.

Operations and Maintenance

Infiltration facility maintenance should include frequent inspections to ensure that surface ponding infiltrates into the subsurface completely within the design infiltration time after a storm (see Appendix I for an infiltration BMP inspection and maintenance checklist).

Maintenance and regular inspections are of primary importance if infiltration BMPs are to continue to function as originally designed. A specific maintenance plan shall be formulated specifically for each facility outlining the schedule and scope of maintenance operations, as well as the data handling and reporting requirements. The following are general maintenance requirements:

1) Regular inspection should determine if the pretreatment sediment removal BMPs require routine maintenance.

2) If water is noticed in the basin more than 72 hours after a major storm the infiltration facility may be clogged. Maintenance activities triggered by a potentially clogged facility include:

   a. Check for debris/sediment accumulation, rake surface, and remove sediment (if any) and evaluate potential sources of sediment and debris (e.g., embankment erosion, channel scour, overhanging trees, etc). If suspected upland sources are outside of the immediate jurisdiction, additional pretreatment operations (e.g., trash racks, vegetated swales, etc.) may be necessary.

   b. For basins, removal of the top layer of native soil may be required to restore infiltrative capacity.

   c. Any debris or algae growth located on top of the infiltration facility should be removed and disposed of properly.

   d. Facilities shall be inspected annually. Trash and debris should be removed as needed, but at least annually prior to the beginning of the wet season.

3) Site vegetation should be maintained as frequently as necessary to maintain the aesthetic appearance of the site, and as follows:

    a. Vegetation, large shrubs, or trees that limit access or interfere with basin operation should be pruned or removed.

    b. Slope areas that have become bare should be revegetated and eroded areas should be regraded prior to being revegetated.

    c. Grass should be mowed to 4" - 9" high and grass clippings should be removed.

    d. Fallen leaves and debris from deciduous plant foliage should be raked and removed.

    e. Invasive vegetation, such as Alligatorweed (Alternanthera philoxeroides), Halogeton (Halogeton glomeratus), Spotted Knapweed (Centaurea maculosa), Giant Reed (Arundo donax), Castor Bean (Ricinus communis), Perennial Pepperweed (Lepidium latifolium), and Yellow Starthistle (Centaurea solstitalis) should be removed and replaced with non-invasive species. Invasive species should never contribute more than 25% of the vegetated area. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website or the California Invasive Plant Council website at www.cal-ipc.org.

    f. Dead vegetation should be removed if it exceeds 10% of area coverage. Vegetation should be replaced immediately to maintain cover density and control erosion where soils are exposed.

4) For infiltration basins, sediment build-up exceeding 50% of the forebay capacity should be removed. Sediment from the remainder of the basin should be removed when 6 inches of sediment accumulates. Sediments should be tested for toxic substance accumulation in compliance with current disposal requirements if land uses in the catchment include commercial or industrial zones, or if visual or olfactory indications of pollution are noticed. If toxic substances are encountered at concentrations exceeding thresholds of Title 22, Section 66261 of the California Code of Regulations, the sediment should be disposed of in a hazardous waste landfill and the source of the contaminated sediments should be investigated and mitigated to the extent possible.

5) Following sediment removal activities, replanting and/or reseeding of vegetation may be required for reestablishment.

INF-2: Infiltration Trench

Infiltration trenches are long, narrow, gravel-filled trenches, often vegetated, that infiltrate stormwater runoff from small drainage areas. Infiltration trenches may include a shallow depression at the surface, but the majority of runoff is stored in the void space within the gravel and infiltrates through the sides and the bottom of the trench.

Rural Highway Infiltration Trench

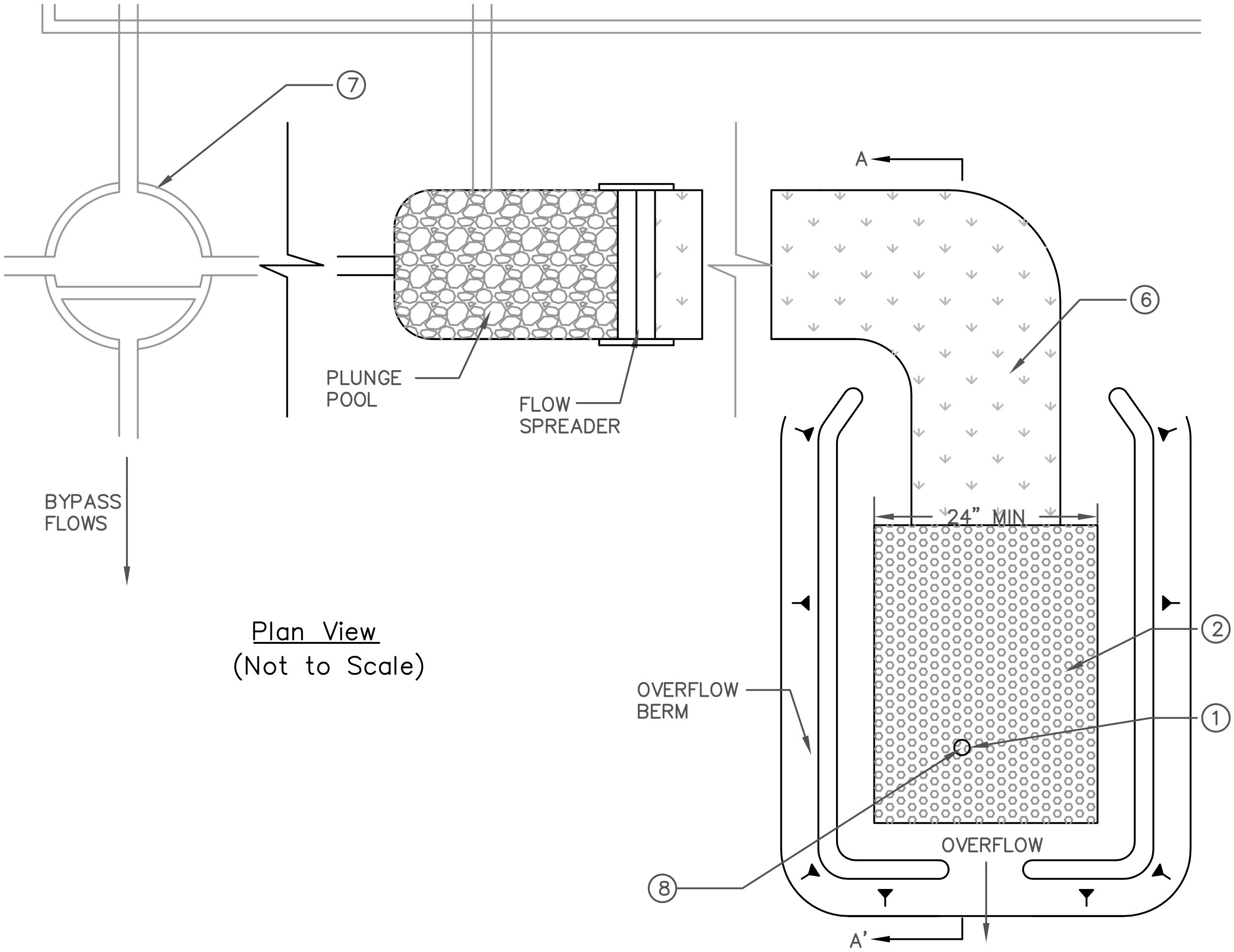
http://stormwater.wordpress.com/2007/05/23/infiltration--trenches/

Application

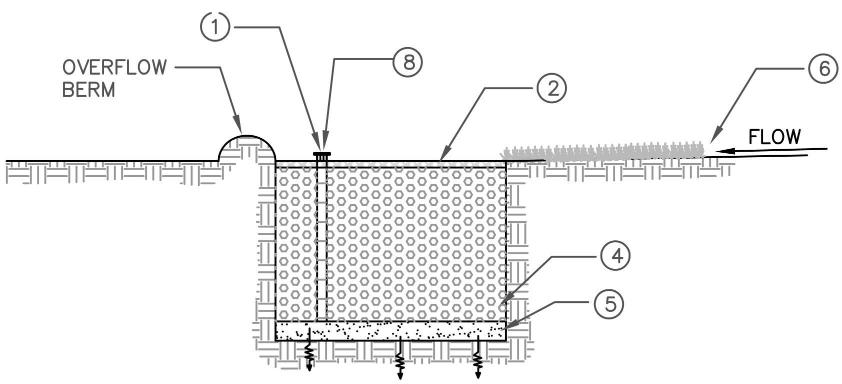
- Open areas adjacent to parking lots, driveways, and buildings
- Roadway medians and shoulders

Routine Maintenance

- Removal trash, debris, and sediment at inlet and outlets
- Wet weather inspection to ensure drain time
- Remove weeds
- Inspect for mosquito breeding



Plan View
(Not to Scale)



Section A – A'
(Not to Scale)

NOTES:

1. OBSERVATION WELL WITH LOCKABLE ABOVE-GROUND CAP.
2. 2" PEA GRAVEL FILTER LAYER.
3. MINIMUM 10' ABOVE SEASONAL HIGH GROUNDWATER TABLE AND 3' ABOVE BEDROCK.
4. 3' – 5' DEEP TRENCH FILLED WITH 2" – 6" DIAMETER CLEAN STONE WITH 30% – 40% VOIDS.
5. 6" DEEP SAND FILTER LAYER (OR FABRIC EQUIVALENT).
6. RUNOFF FILTERS THROUGH GRASS FILTER STRIP OR VEGETATED SWALE.
7. OPTIONAL FLOW CONTROL DEVICE FOR OFF-LINE CONFIGURATIONS.

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Figure 6-3: Infiltration Trench

Limitations

The following limitations should be considered before choosing to use an infiltration trench:

- Native soil infiltration rate – soil permeability at the infiltration trench location must be at least 0.5 inches per hour.

- Depth to groundwater, bedrock, or low permeability soil layer – 5 feet vertical separation is required between the bottom of the infiltration trench and the seasonal high groundwater level or mounded groundwater level, bedrock, or other barrier to infiltration to ensure that the facility will completely drain between storms and that infiltrating water will receive adequate treatment though the soils before it reaches the groundwater.

- Slope stability - infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.

- Setbacks - a minimum setback (100 feet or more) must be provided between infiltration BMPs and potable wells, non-potable wells, drain fields and springs. Infiltration BMPs must be setback from building foundations at least eight feet or an alternative setback established by the geotechnical expert for the project.

- Groundwater contamination - the application of infiltration BMPs should include significant pretreatment in an area identified as an unconfined aquifer to ensure groundwater is protected for pollutants of concern.

- Contaminated soils or groundwater plumes - infiltration BMPs are not allowed at locations with contaminated soils or groundwater where the pollutants could be mobilized or exacerbated by infiltration, unless a site-specific analysis determines that infiltration would be beneficial.

- High pollutant land uses - infiltration BMPs should not be placed in high-risk areas such as at or near service/gas stations, truck stops, and heavy industrial sites due to the groundwater contamination risk unless a site-specific evaluation demonstrates that sufficient pretreatment is provided to address pollutants of concern, high risks areas are isolated from stormwater runoff, or infiltration areas have little chance of spill migration.

- High sediment loading rates – infiltration BMPs may clog quickly if sediment loads are high (e.g., unstabilized site) or if flows are not adequately pretreated.

Design Criteria

The main challenge associated with infiltration trenches is preventing system clogging and subsequent infiltration inhibition. Infiltration trenches should be designed according to the requirements listed in Table 6-5 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-5: Infiltration Trench Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume (SQDV)	acre-feet	See Section 2 and Appendix E for calculating SQDV.
Design drawdown time	hr	12 – 72, see Appendix D, Section D.2
Trench bottom elevation	feet	5 feet from seasonally high groundwater table
Setbacks	feet	100 feet from wells, fields, springs Geotechnical expert should establish the setback requirement from building foundations that must be ≥ 8 ft Do not locate under tree drip-lines
Pretreatment	-	BIO-3: Vegetated Swale, BIO-4: Filter Strip, proprietary device, or sedimentation forebay, for all surfaces other than roofs
Design percolation rate, (P_{design})	in/hr	Measured percolation rate must be corrected based onsite suitability assessment and design related considerations described in this fact sheet
Maximum depth of facility (d_{max})	feet	8.0; Defined by the design infiltration rate and the design drawdown time (includes ponding depth and depth of media)
Surface area of facility (A)	square feet	Based on depth of ponding (if applicable) and depth of trench media
Facility geometry	-	Minimum 24 inches wide and maximum 5 feet deep; max 3% bottom slope
Filter media diameter	inches	1 – 3 (gravel); prefabricated media may also be used
Trench lining material	-	Geotextile fabric
Overflow device	-	Required if system is on-line

Geotechnical Considerations

An extensive geotechnical site investigation must be undertaken early in the site planning process to verify site suitability for the installation of infiltration facilities due to the potential to contaminate groundwater, cause slope instability, impact surrounding structures, and have insufficient infiltration capacity. Soil infiltration rates and the water table depth should be evaluated to ensure that conditions are satisfactory for proper operation of an infiltration facility. See Appendix C for guidance on infiltration testing.

The project designer must demonstrate through infiltration testing, soil logs, and the written opinion of a licensed civil engineer that sufficiently permeable soils exist onsite to allow the construction of a properly functioning infiltration facility.

1) Infiltration facilities require a minimum soil infiltration rate of 0.5 inches/hour. If infiltration rates exceed 2.4 inches/hour, then the runoff should be fully treated in an upstream BMP prior to infiltration to protect groundwater quality. Pretreatment for coarse sediment removal is required in all instances.

2) Groundwater separation must be at least 5 feet from the trench bottom to the measured season high groundwater elevation or estimated high groundwater mounding elevation. Groundwater level measurements must be made during the time when water level is expected to be at a maximum (i.e., toward the end of the wet season).

3) Sites with a slope greater than 25% (4:1) should be excluded. A geotechnical analysis and report addressing slope stability are required if located on slopes greater than 15%.

Soil Assessment and Site Geotechnical Investigation Reports

The soil assessment report should:

- State whether the site is suitable for the proposed infiltration trench;
- Recommend a design infiltration rate (see the Step 2 of sizing methodology section, “Determine the design percolation rate,” in the Infiltration Basin fact sheet above);
- Identify the seasonally high depth to groundwater table surface elevation.
- Provide a good understanding of how the stormwater runoff will move in the soil (horizontally or vertically) and if there are any geological conditions that could inhibit the movement of water; and
- If a geotechnical investigation and report are required, the report should:
  - Provide a written opinion by a professional civil engineer describing whether the infiltration trench will compromise slope stability; and

- Identify potential impacts to nearby structural foundations.

Setbacks

1) Infiltration facilities shall be setback a minimum of 100 feet from proposed or existing potable wells, non-potable wells, septic drain fields, and springs.

2) Infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.

3) Infiltration BMPs must be setback from building foundations at least eight feet or an alternative setback established by the geotechnical expert for the project.

Pretreatment

Pretreatment is required for infiltration trenches in order to reduce the sediment load entering the facility and maintain the infiltration rate of the facility. Pretreatment refers to design features that provide settling of large particles before runoff reaches a management practice; easing the long-term maintenance burden. Pretreatment is important for most all structural stormwater BMPs, but it is particularly important for infiltration BMPs. To ensure that pretreatment mechanisms are effective, designers should incorporate sediment reduction practices. Sediment reduction BMPs may include vegetated swales, vegetated filter strips, sedimentation basins or forebays, sedimentation manholes and hydrodynamic separation devices.

For design specification of selected pre-treatment devices, refer to:

- VEG-3: Vegetated swales
- VEG-4: Vegetated filter strips
- TCM-4: Sand filters
- TCM-5: Cartridge media filters
- PT-1: Hydrodynamic separation device

Sizing Criteria

See Sizing Criteria section in the INF-1: Infiltration Basin fact sheet.

Geometry and Sizing

1) Infiltration trenches should be at least 2 feet wide and 3 to 5 feet deep.

2) The longitudinal slope of the trench should not exceed 3%.

3) The filter bed media layers should have the following composition and thickness:

a. Top layer – If stormwater runoff enters the top of the trench via sheet flow at the ground surface, then the top 2 inches should be pea gravel with a thin 2 to 4 inch layer of pure sand and 2 inch layer of chocking stone (e.g., #8) to capture sediment before entering the trench. If stormwater runoff enters the trench from an underground pipe, pretreatment prior to entry into the trench is required.

b. Middle layer (3 to 5 feet of washed, 1.5 to 3 inch gravel). Void space should be in the range of 30 percent to 40 percent.

c. Bottom layer (6 inches of clean, washed sand to encourage drainage and prevent compaction of the native soil while the stone aggregate is added).

4) One or more observation wells should be installed, depending on trench length, to check for water level, drawdown time, and evidence of clogging. A typical observation well consists of a slotted PVC well screen, 4 to 6 inches in diameter, capped with a lockable, above-ground lid.

Drainage

1) The bottom of the infiltration bed must be native soil, over-excavated to at least one foot in depth and replaced uniformly without compaction. Amending the excavated soil with 2 to 4 inches (~15% to 30%) of coarse sand is recommended.

2) The hydraulic conductivity of the subsurface layers should be sufficient to ensure the design drawdown time. An observation well should be incorporated to allow observation of drain time.

Emergency Overflow

1) There must be an overflow route for stormwater flows that overtop the facility or in case the infiltration facility becomes clogged.

2) The overflow channel must be able to safely convey flows from the peak design storm to the downstream stormwater conveyance system or other acceptable discharge point.

Vegetation

1) Trees and other large vegetation should be planted away from trenches such that drip lines do not overhang infiltration beds.

Maintenance Access

1) The facility and outlet structures must all be safely accessible during wet and dry weather conditions.

2) An access road along the length of the trench is required, unless the trench is located along an existing road or parking lot that can be safely used for maintenance access.

3) If the infiltration trench becomes plugged and fails, then access is needed to excavate the facility to remove and replace the top layer or the filter bed media, as well as to increase all dimensions of the facility by 2 inches to provide a fresh surface for infiltration. To prevent damage and compaction, access must be able to accommodate a backhoe working at “arms length”.

Construction Considerations

To preserve and avoid the loss of infiltration capacity, the following construction guidelines are specified:

1) The entire area draining to the facility must be stabilized before construction begins. If this is impossible, a diversion berm should be placed around the perimeter of the infiltration site to prevent sediment entering during construction.

2) Infiltration trenches should not be hydraulically connected to the stormwater conveyance system until all contributing tributary areas are stabilized as shown on the Contract Plans and to the satisfaction of the Engineer. Infiltration trenches should not be used as sediment control facilities.

3) Compaction of the subgrade with heavy equipment should be minimized to the maximum extent possible. If the use of heavy equipment on the base of the facility cannot be avoided, the infiltrative capacity should be restored by tilling or aerating prior to placing the infiltrative bed.

4) The exposed soils should be inspected by a civil engineer after excavation to confirm that soil conditions are suitable.

Operations and Maintenance

Infiltration facility maintenance should include frequent inspections to ensure that water infiltrates into the subsurface completely within the design drawdown time after a storm.

Maintenance and regular inspections are of primary importance if infiltration trenches are to continue to function as originally designed. A specific maintenance plan shall be developed specific to each facility outlining the schedule and scope of maintenance operations, as well as the documentation and reporting requirements. The following are general maintenance requirements:

1) Regular inspection should determine if the sediment pretreatment structures require preventative maintenance. Inspect a minimum of twice a year, before and after the rainy season, after large storms, or more frequently if needed.

2) If water is noticed in the observation well of the infiltration trench more than 72 hours after a major storm, the infiltration trench may be clogged. Maintenance activities triggered by a potentially clogged facility include:

a. For trenches, assess the condition of the top aggregate layer for sediment buildup and crusting. Remove top layer of pea gravel and replace. If slow draining conditions persist, entire trench may need to be excavated and replaced.

3) Any debris or algae growth located on top of the infiltration facility should be removed and disposed of properly.

4) Inspect a minimum of twice a year, before and after the rainy season, after large storms, or more frequently if needed.

5) Clean when loss of infiltrative capacity is observed. If drawdown time is observed to have increased significantly over the design drawdown time, removal of sediment may be necessary. This is an expensive maintenance activity and the need for it can be minimized through prevention of upstream erosion.

6) Mow as appropriate for vegetative cover species.

7) Monitor health of vegetation and replace as necessary.

8) Control mosquitoes as necessary.

9) Remove litter and debris from trench area as required.

INF-3: Bioretention

Bioretention stormwater treatment facilities are landscaped shallow depressions that capture and filter stormwater runoff. These facilities function as a soil and plant-based filtration device that removes pollutants through a variety of physical, biological, and chemical treatment processes. The facilities normally consist of a ponding area, mulch layer, planting soils, and plantings. An optional gravel layer can be added below the planting soil to provide additional storage volume for infiltration. As stormwater passes down through the planting soil, pollutants are filtered, adsorbed, and biodegraded by the soil and plants. For areas with low permeability native soils or steep slopes, see section INF-7: Bioinfiltration or BIO-1: Bioretention with Underdrain for relevant design specifications.

Bioretention in Parkway and parking lots

Photo Credits: Geosyntec Consultants

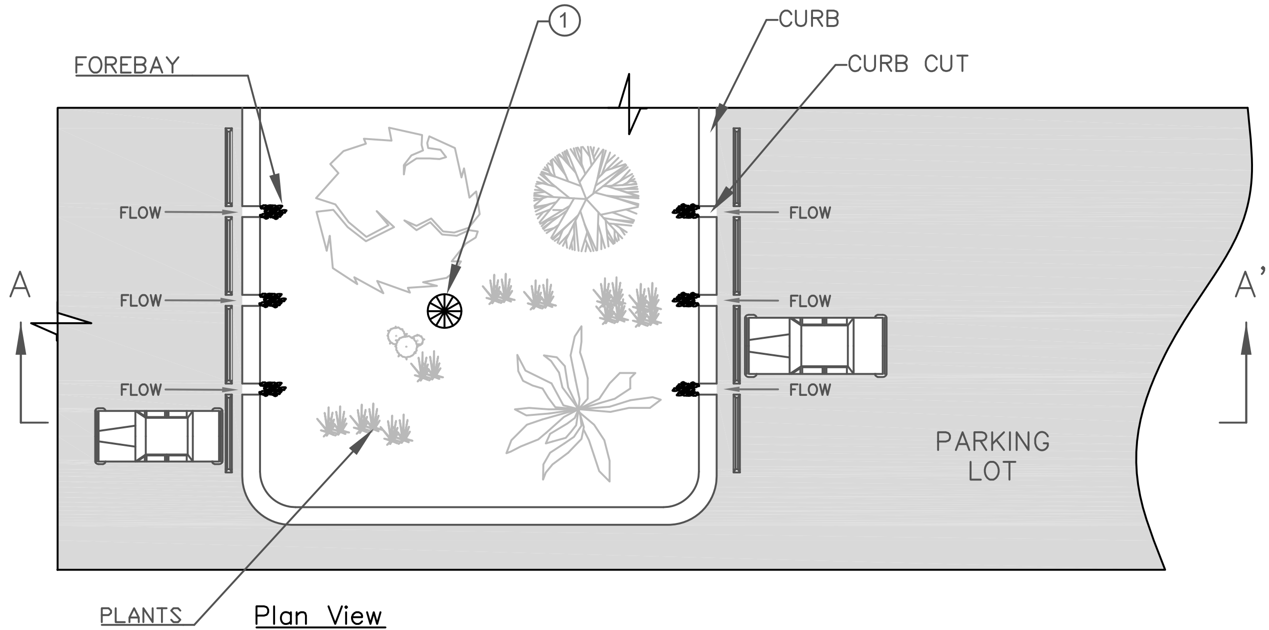
Application

- Commercial, residential, mixed use, institutional, and recreational uses
- Parking lot islands, traffic circles
- Road parkways & medians

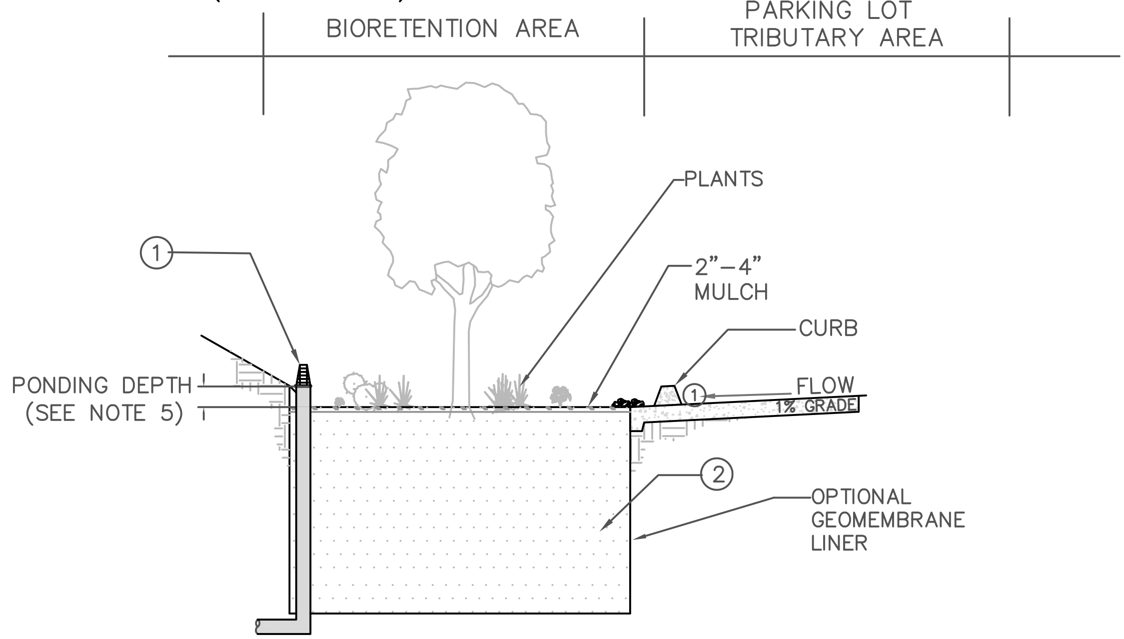
Preventative Maintenance

- Repair small eroded areas
- Remove trash and debris and rake surface soils
- Remove accumulated fine sediments, dead leaves and trash
- Remove weeds and prune back excess plant growth
- Remove sediment and debris accumulation near inlet and outlet structures
- Periodically observe function under wet weather conditions

Plan View
(Not to Scale)



Plan View
(Not to Scale)



Section A – A'
(Not to Scale)

NOTES:

1. OVERFLOW DEVICE: VERTICAL RISER OR EQUIVALENT.
2. 2' MIN PLANTING MIX; 3' PREFERRED.
3. PONDING DEPTH 18" WITH FENCE; 6" WITHOUT FENCE.

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Figure 6-4: Bioretention

Limitations

The following limitations should be considered before choosing to use bioretention:

1) Native soil infiltration rate - soil permeability at the bioretention location must be at least 0.5 inches per hour.

2) Depth to groundwater, bedrock, or low permeability soil layer – 5 feet vertical separation is required between the bottom of the infiltration trench and the seasonal high groundwater level or mounded groundwater level, bedrock, or other barrier to infiltration to ensure that the facility will completely drain between storms and that infiltrating water will receive adequate treatment though the soils before it reaches the groundwater.

3) Slope stability - infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.

4) Setbacks - a minimum setback (100 feet or more) must be provided between infiltration BMPs and potable wells, non-potable wells, drain fields, and springs. Infiltration BMPs must be setback from building foundations at least eight feet or have an alternative setback established by the geotechnical expert for the project.

5) Groundwater contamination - the application of infiltration BMPs should include significant pretreatment in an area identified as an unconfined aquifer to ensure groundwater is protected for pollutants of concern.

6) Contaminated soils or groundwater plumes - infiltration BMPs are not allowed at locations with contaminated soils or groundwater where the pollutants could be mobilized or exacerbated by infiltration, unless a site-specific analysis determines that infiltration would be beneficial.

7) High pollutant land uses - infiltration BMPs should not be placed in high-risk areas such as at or near service/gas stations, truck stops, and heavy industrial sites due to the groundwater contamination risk unless a site-specific evaluation demonstrates that sufficient pretreatment is provided to address pollutants of concern, high risks areas are isolated from stormwater runoff, or infiltration areas have little chance of spill migration.

8) High sediment loading rates – infiltration BMPs may clog quickly if sediment loads are high (e.g., unstabilized site) or if flows are not adequately pretreated.

9) Vertical relief and proximity to storm drain - site must have adequate relief between the land surface and storm drain to permit vertical percolation through the soil media and collection.

Design Criteria

Bioretention should be designed according to the requirements listed in Table 6-6 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-6: Bioretention Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume (SQDV)	acre-feet	See Section 2 and Appendix E for calculating SQDV.
Forebay	-	Forebay should be provided for all tributary surfaces that contain landscaped areas. Forebays should be designed to prevent standing water during dry weather and should be planted with a plant palette that is tolerant of wet conditions.
Maximum drawdown time of water ponded on surface	hours	48
Maximum drawdown time of surface ponding plus subsurface pores	hours	96 (72 preferred)
Maximum ponding depth	inches	18
Minimum thickness of amended soil	feet	2 (3 preferred)
Minimum thickness of stabilized mulch	inches	2 to 3
Planting mix composition	-	60 to 80% fine sand, 20 to 40% compost
Overflow device	-	Required

Sizing Criteria

Bioretention facilities can be sized using one of two methods: a simple sizing method or a routing modeling method. With either method the SQDV volume must be completely infiltrated within 96 hours (including subsurface pore space), and surface ponding must be infiltrated within 48 hours. The simple sizing procedure is provided below. For the routing modeling method, refer to TCM-4 Sand Filters.

Step 1: Calculate the Design Volume

Bioretention facilities shall be sized to capture and infiltrate the SQDV volume (see Section 2.3 and Appendix E).

Step 2: Determine the Design Percolation Rate

The percolation rate through the BMP and to the subsurface will decline between maintenance cycles as the surface becomes occluded and particulates accumulate in the infiltration layer. Monitoring of actual facility performance has shown that the full-scale infiltration rate is far lower than the rate measured by small-scale testing. It is important that adequate conservatism is incorporated in the selection of design percolation rates. For bioretention facilities, the design percolation rate discussed here is the adjusted percolation rate of the underlying soils and not the percolation rate of the filter media bed.

<u>Considerations for Design Percolation Rate Corrections</u>

Suitability assessment-related considerations include (Table 6-7):

- Soil assessment methods – the site assessment extent (e.g., number of borings, test pits, etc.) and the measurement method used to estimate the short-term infiltration rate.
- Predominant soil texture/percent fines – soil texture and the percent of fines can greatly influence the potential for clogging.
- Site soil variability – site with spatially heterogeneous soils (vertically or horizontally) as determined from site investigations are more difficult to estimate average properties, resulting in a higher level of uncertainty associated with initial estimates.
- Depth to seasonal high groundwater/impervious layer – groundwater mounding may become an issue during excessively wet conditions where shallow aquifers or shallow clay lenses are present.

Localized infiltration testing refers to methods such as the double ring infiltrometer test (ASTM D3385-88), which measure infiltration rates over an area less than 10 sq-ft and do not attempt to account for soil heterogeneity. Extensive infiltration testing refers to methods that include excavating a significant portion of the proposed infiltration area, filling the excavation with water, and monitoring drawdown. In all cases, testing should be conducted in the area of the proposed BMP where, based on geotechnical data, soils appear least likely to support infiltration.

Table 6-7: Suitability Assessment Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern	Medium Concern	Low Concern
Assessment methods	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates	Direct measurement of ≥ 20 percent of infiltration area with localized infiltration measurement methods (e.g., infiltrometer)	Direct measurement of ≥ 50 percent of infiltration area with localized infiltration measurement methods or Use of extensive test pit infiltration measurement methods
Ventura Hydrology Manual soil number (measured infiltration rate)	3 (f = 0.5 – 0.64)	4 or 5 (f = 0.65 – 0.91)	6 or 7 (f = 0.92 or higher)
Site soil variability	Highly variable soils indicated from site assessment or limited soil borings collected during site assessment	Soil borings/test pits indicate moderately homogeneous soils	Multiple soil borings/test pits indicate relatively homogeneous soils
Depth to groundwater/ impervious layer	<10 ft below facility bottom	10-30 ft below facility bottom	>30 below facility bottom

Design related considerations include:

- Size of area tributary to facility – all things being equal, both physical and economic risk factors related to infiltration facilities increase with an increase in the tributary area served. Therefore facilities serving larger tributary areas should use more restrictive adjustment factors.

- Level of pretreatment/expected influent sediment loads – credit should be given for good pretreatment by allowing less restrictive factors to account for the reduced probability of clogging from high sediment loading. Also, facilities designed to capture runoff from relatively clean surfaces such as rooftops are likely to see low sediment loads and therefore should be allowed to apply less restrictive safety factors.

- Redundancy – facilities that consist of multiple subsystems operating in parallel such that parts of the system remain functional when other parts fail and/or bypass should be rewarded for the built-in redundancy with less restrictive correction and safety factors. For example, if bypass flows would be at least partially treated in another BMP, the risk of discharging untreated runoff in the event of clogging the primary facility is reduced. A bioretention facility that overflows to a landscaped area is another example.

- Compaction during construction – proper construction oversight is needed during construction to ensure that the bottoms of bioretention facility are not overly compacted. Facilities that do not commit to proper construction practices and oversight should have to use more restrictive correction and safety factors.

Table 6-8: Design Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern	Medium Concern	Low Concern
Tributary area size	Greater than 10 acres.	Greater than 2 acres but less than 10 acres.	2 acres or less.
Level of pre-treatment/ expected influent sediment loads	Pre-treatment from gross solids removal devices only, such as hydrodynamic separators, racks and screens, AND tributary area includes landscaped areas, steep slopes, high traffic areas, or any other areas expected to produce high sediment, trash, or debris loads.	Good pre-treatment with BMPs that mitigate coarse sediments such as vegetated swales AND influent sediment loads from the tributary area are expected to be relatively low (e.g., low traffic, mild slopes, disconnected impervious areas, etc.).	Excellent pre-treatment with BMPs that mitigate fine sediments such as bioretention or media filtration OR sedimentation or facility only treats runoff from relatively clean surfaces, such as rooftops.
Redundancy of treatment	No redundancy in BMP treatment train.	Medium redundancy, other BMPs available in treatment train to maintain at least 50% of function of facility in event of failure.	High redundancy, multiple components capable of operating independently and in parallel, maintaining at least 90% of facility functionality in event of failure.
Compaction during construction	Construction of facility on a compacted site or elevated probability of unintended/ indirect compaction.	Medium probability of unintended/ indirect compaction.	Heavy equipment actively prohibited from infiltration areas during construction and low probability of unintended/ indirect compaction.

Adjust the measured short-term infiltration rate using a weighted average of several safety factors using the worksheet shown in Table 6-9 below. The design percolation rate would be determined as follows:

- For each consideration shown in Tables 6-7 and 6-8 above, determine whether the consideration is a high, medium, or low concern.
- For all high concerns assign a factor value of 3, for medium concerns assign a factor value of 2, and for low concerns assign a factor value of 1.
- Multiply each of the factors by the corresponding weight to get a product.
- Sum the products within each factor category to obtain a safety factor for each.
- Multiply the two safety factors together to get the final combined safety factor. If the combined safety factor is less than 2, then use 2 as the safety factor.
- Divide the measured short-term infiltration rate by the combined safety factor to obtain the adjusted design percolation rate for use in sizing the infiltration facility.

Table 6-9: Infiltration Facility Safety Factor Determination Worksheet

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) p = w x v
A	Suitability Assessment	Soil assessment methods	0.25		
		Predominant soil texture	0.25		
		Site soil variability	0.25		
		Depth to groundwater / impervious layer	0.25		
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			
B	Design	Tributary area size	0.25		
		Level of pre-treatment/ expected sediment loads	0.25		
		Redundancy	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \Sigma p$			
Combined Safety Factor = $S_A \times S_B$					

Note: The minimum combined adjustment factor shall not be less than 2.0 and the maximum combined adjustment factor shall not exceed 9.

Step 3: Calculate the surface area

Determine the size of the required infiltrating surface by assuming the SQDV will fill the available ponding depth plus the void spaces in the media, based on the computed porosity of the filter media and optional aggregate layer.

1) Determine the maximum depth of surface ponding that can be infiltrated within the required surface drain time (48 hr), (d_{max}), as follows:

$$d_{\max} = \frac{P_{design} \times t_{ponding}}{12 \frac{in}{ft}}$$ (Equation 6-6)

Where:

$t_{ponding}$ = required drain time of surface ponding (48 hrs)

P_{design} = design percolation rate of underlying soils (in/hr) (see Step 2, above)

d_{max} = the maximum depth of surface ponding water that can be infiltrated within the required drain time (ft), calculated using Equation 6-6

2) Choose surface ponding depth (d_p) such that:

$$d_p \le d_{\max}$$ (Equation 6-7)

Where:

d_p = selected surface ponding depth (ft)

d_{max} = the maximum depth of water that can be infiltrated within the required drain time (ft)

Choose thickness(es) of amended media and optional gravel storage layer and calculate total effective storage depth of the bioretention area ($d_{effective}$), as follows:

$$d_{effective} \le (d_p + n^*_{media} l_{media} + n_{gravel} l_{gravel})$$ (Equation 6-8)

Where:

$d_{effective}$ = total equivalent depth of water stored in bioretention area (ft), including surface ponding and volume available in pore spaces of media and gravel layers

d_p = surface ponding depth (ft), chosen using Equation 6-7

n^*_{media} = available porosity of amended soil media (ft/ft), approximately 0.25 ft/ft accounting for antecedent moisture conditions. This represents the volume of available pore space as a fraction of the total soil volume; sometimes has units of (ft^3/ft^3) or described as a percentage.

l_{media} = thickness of amended soil media layer (ft), minimum 2 ft

n_{gravel} = porosity of optional gravel layer (ft/ft), approximately 0.40 ft/ft

l_{gravel} = thickness of optional gravel layer (ft)

3) Check that entire effective depth (surface plus subsurface storage), $d_{effective}$, infiltrates in no greater than 96 hours as follows:

$$t_{total} = \frac{d_{effective}}{P_{design}} \times 12 \frac{in}{ft} \leq 96\ hr \quad \text{(Equation 6-9)}$$

Where:

$d_{effective}$ = total equivalent depth of water stored in bioretention area (ft), calculated using Equation 6-8

P_{design} = design percolation rate of underlying soils (in/hr) (see Step 2, above)

If $t_{total} > 96$ hrs, then reduce surface ponding depth and/or amended media thickness and/or gravel thickness and return to 1).

If $t_{total} \leq 96$ hrs, then proceed to 5).

4) Calculate required infiltrating surface area, (A_{req}):

$$A_{req} = \frac{SQDV}{d_{effective}} \quad \text{(Equation 6-10)}$$

Where:

A_{req} = required infiltrating area (ft^2). Should be calculated at the contour corresponding to the mid ponding depth (i.e., $0.5 \times d_p$ from the bottom of the facility).

$SQDV$ = stormwater quality design volume (ft^3)

$d_{effective}$ = total equivalent depth of water stored in bioretention area (ft), calculated using Equation 6-8

5) Calculate total footprint required by including a buffer for side slopes and freeboard; A_{req} is calculated at the contour corresponding to the mid ponding depth (i.e., $0.5\times d_p$ from the bottom of the facility).

Geometry

1) Bioretention areas shall be sized to capture and treat the stormwater quality design volume (See Section 2 and Appendix E for calculating SQDV) with an 18-inch maximum ponding depth. *The intention is that ponding depth be limited to a depth that will allow for a health vegetation layer.*

2) Minimum planting soil depth should be 2 feet, although 3 feet is preferred. *The intention is that the minimum planting soil depth should provide a beneficial root zone for the chosen plant palette and adequate water storage for the SQDV.*

3) A gravel storage layer below the bioretention soil media to promote infiltration into the native soil is optional.

4) Bioretention should be designed to drain below the planting soil in less than 48 hours and completely drain in less than 96 hours. *The intention is that soils must be allowed to dry out periodically in order to restore hydraulic capacity needed to receive flows from subsequent storms, maintain infiltration rates, maintain adequate soil oxygen levels for healthy soil biota and vegetation, and to provide proper soil conditions for biodegradation and retention of pollutants.*

Flow Entrance and Energy Dissipation

The following types of flow entrance can be used for bioretention cells:

1) Dispersed, low velocity flow across a landscape area. Dispersed flow may not be possible given space limitations or if the facility is controlling roadway or parking lot flows where curbs are mandatory.

2) Dispersed flow across pavement or gravel and past wheel stops for parking areas.

3) Curb cuts for roadside or parking lot areas: curb cuts should include rock or other erosion protection material in the channel entrance to dissipate energy. Flow entrance should drop 2 to 3 inches from curb line and it should provide a settling area and periodic sediment removal of coarse material before flow dissipates to the remainder of the cell.

4) Pipe flow entrance: Piped entrances, such as roof downspouts, should include rock, splash blocks, or other appropriate measures at the entrance to dissipate energy and disperse flows.

Woody plants (trees, shrubs, etc.) can restrict or concentrate flows and can be damaged by erosion around the root ball and should not be placed directly in the entrance flow path.

Overflow

An overflow device is required at the 18-inch ponding depth. The following, or equivalent should be provided:

1) A vertical PVC pipe (SDR 35) to act as an overflow riser.

2) The overflow riser(s) should be 6 inches or greater in diameter, so it can be cleaned without damage to the pipe.

The inlet to the riser should be at the ponding depth (18 inches for fenced bioretention areas and 6 inches for areas that are not fenced), and be capped with a spider cap to exclude floating mulch and debris. Spider caps should be screwed in or glued, i.e., not removable.

Hydraulic Restriction Layers

Infiltration pathways may need to be restricted due to the close proximity of roads, foundations, or other infrastructure. A geomembrane liner, or other equivalent water proofing, may be placed along the vertical walls to reduce lateral flows. This liner should have a minimum thickness of 30 mils.

Planting/Storage Media

1) The planting media placed in the cell should achieve a long-term, in-place infiltration rate of at least 1 inch per hour. Higher infiltration rates are permissible. If the design long-term, in-place infiltration rate of the soil exceeds 12 inches per hour, documentation should be provided to demonstrate that the media will adequately address pollutants of concern at a higher flowrate. Bioretention soil shall also support vigorous plant growth.

2) Planting media should consist of 60 to 80% fine sand and 20 to 40% compost.

3) Sand should be free of wood, waste, coating such as clay, stone dust, carbonate, etc., or any other deleterious material. All aggregate passing the No. 200 sieve size should be non-plastic. Sand for bioretention should be analyzed by an accredited lab using #200, #100, #40, #30, #16, #8, #4, and 3/8 sieves (ASTM D 422 or as approved by the local permitting authority) and meet the following gradation (Note: all sands complying with ASTM C33 for fine aggregate comply with the gradation requirements below):

Sieve Size (ASTM D422)	% Passing (by weight)	
	Minimum	Maximum
3/8 inch	100	100
#4	90	100
#8	70	100
#16	40	95
#30	15	70
#40	5	55
#100	0	15
#200	0	5

Note: the gradation of the sand component of the media is believed to be a major factor in the hydraulic conductivity of the media mix. If the desired hydraulic conductivity of the media cannot be achieved within the specified proportions of sand and compost (#2), then it may be necessary to utilize sand at the coarser end of the range specified in above ("minimum" column).

4) Compost should be a well decomposed, stable, weed free organic matter source derived from waste materials including yard debris, wood wastes, or other organic materials not including manure or biosolids meeting standards developed by the US Composting Council (USCC). The product shall be certified through the USCC Seal of Testing Assurance (STA) Program (a compost testing and information disclosure program). Compost quality should be verified via a lab analysis to be:

- Feedstock materials shall be specified and include one or more of the following: landscape/yard trimmings, grass clippings, food scraps, and agricultural crop residues.
- Organic matter: 35-75% dry weight basis.
- Carbon and Nitrogen Ratio: 15:1 < C:N < 25:1
- Maturity/Stability: shall have dark brown color and a soil-like odor. Compost exhibiting a sour or putrid smell, containing recognizable grass or leaves, or is hot (120 F) upon delivery or rewetting is not acceptable.
- Toxicity: any one of the following measures is sufficient to indicate non-toxicity:
    - NH_4:NH_3 < 3
    - Ammonium < 500 ppm, dry weight basis
    - Seed Germination > 80% of control
    - Plant trials > 80% of control

    - Solvita® > 5 index value

- Nutrient content:

    - Total Nitrogen content 0.9% or above preferred

    - Total Boron should be <80 ppm, soluble boron < 2.5 ppm

- Salinity: < 6.0 mmhos/cm

- pH between 6.5 and 8 (may vary with plant palette)

Compost for bioretention should be analyzed by an accredited lab using #200, ¼ inch, ½ inch, and 1 inch sieves (ASTM D 422 or as approved by the local permitting authority) and meet the following gradation:

Sieve Size (ASTM D422)	% Passing (by weight)	
	Minimum	Maximum
1 inch	99	100
½ inch	90	100
¼ inch	40	90
#200	2	10

Tests should be sufficiently recent to represent the actual material that is anticipated to be delivered to the site. If processes or sources used by the supplier have changed significantly since the most recent testing, new tests should be requested.

Note: the gradation of compost used in bioretention media is believed to play an important role in the saturated hydraulic conductivity of the media. To achieve a higher saturated hydraulic conductivity, it may be necessary to utilize compost at the coarser end of this range ("minimum" column). The percent passing the #200 sieve (fines) is believed to be the most important factor in hydraulic conductivity.

In addition, a coarser compost mix provides more heterogeneity of the bioretention media, which is believed to be advantageous for more rapid development of soil structure needed to support health biological processes. This may be an advantage for plant establishment with lower nutrient and water input.

5) The bioretention area should be covered with 2 to 4 inches (average 3 inches) of mulch at the start and an additional placement of 1 to 2 inches of mulch should be added annually. *The intention is that to help sustain the nutrient levels, suppress weeds, retain moisture, and maintain infiltration capacity*.

Plants

1) Plant materials should be tolerant of summer drought, ponding fluctuations, and saturated soil conditions for 48 to 96 hours.

2) It is recommended that a minimum of three types of tree, shrubs, and/or herbaceous groundcover species be incorporated to protect against facility failure due to disease and insect infestations of a single species.

3) Native plant species and/or hardy cultivars that are not invasive and do not require chemical inputs should be used to the maximum extent practicable.

Operations and Maintenance

Bioretention areas require annual plant, soil, and mulch layer maintenance to ensure optimum infiltration, storage, and pollutant removal capabilities. In general, bioretention maintenance requirements are typical landscape care procedures and include:

1) Watering: Plants should be drought-tolerant. Watering may be required during prolonged dry periods after plants are established.

2) Erosion control: Inspect flow entrances, ponding area, and surface overflow areas periodically, and replace soil, plant material, and/or mulch layer in areas if erosion has occurred (see Appendix I for a bioretention inspection and maintenance checklist). Properly designed facilities with appropriate flow velocities should not have erosion problems, except perhaps in extreme events. If erosion problems occur, the following should be reassessed: (1) flow velocities and gradients within the cell, and (2) flow dissipation and erosion protection strategies in the pretreatment area and flow entrance. If sediment is deposited in the bioretention area, immediately determine the source within the contributing area, stabilize, and remove excess surface deposits.

3) Plant material: Depending on aesthetic requirements, occasional pruning and removing of dead plant material may be necessary. Replace all dead plants and if specific plants have a high mortality rate, assess the cause and, if necessary, replace with more appropriate species. Periodic weeding is necessary until plants are established. The weeding schedule should become less frequent if the appropriate plant species and planting density have been used and, as a result, undesirable plants excluded.

4) Nutrients and pesticides: The soil mix and plants should be selected for optimum fertility, plant establishment, and growth. Nutrient and pesticide inputs should not be required and may degrade the pollutant processing capability of the bioretention area, as well as contribute pollutant loads to receiving waters. By design, bioretention facilities are located in areas where phosphorous and nitrogen levels are often elevated and these should not be limiting nutrients. If in question, have soil analyzed for fertility.

5) Mulch: Replace mulch annually in bioretention facilities where heavy metal deposition is likely (e.g., contributing areas that include industrial and auto dealer/repair parking lots and roads). In residential lots or other areas where metal

deposition is not a concern, replace or add mulch as needed to maintain a 2 to 3 inch depth at least once every two years.

6) Soil: Soil mixes for bioretention facilities are designed to maintain long-term fertility and pollutant processing capability. Estimates from metal attenuation research suggest that metal accumulation should not present an environmental concern for at least 20 years in bioretention systems. Replacing mulch in bioretention facilities where heavy metal deposition is likely provides an additional level of protection for prolonged performance. If in question, have soil analyzed for fertility and pollutant levels.

INF-4: Drywell

A dry well is defined as a bored, drilled, or driven shaft or hole whose depth is greater than its width. A dry well is designed specifically for flood alleviation and stormwater disposal. Drywells are similar to infiltration trenches in their design and function, as they are designed to temporarily store and infiltrate runoff, primarily from rooftops or other impervious areas with low pollutant loading. A dry well may be either a small excavated pit filled with aggregate or a prefabricated storage chamber or pipe segment.

Dry wells can be used to reduce the increased volume of stormwater runoff caused by roofs of buildings. While generally not a significant source of runoff pollution, roofs are one of the most important sources of new or increased runoff volume from land development sites. Dry wells can also be used to indirectly enhance water quality by reducing the amount of SQDV to be treated by the other, downstream stormwater management facilities.

Drywell installation

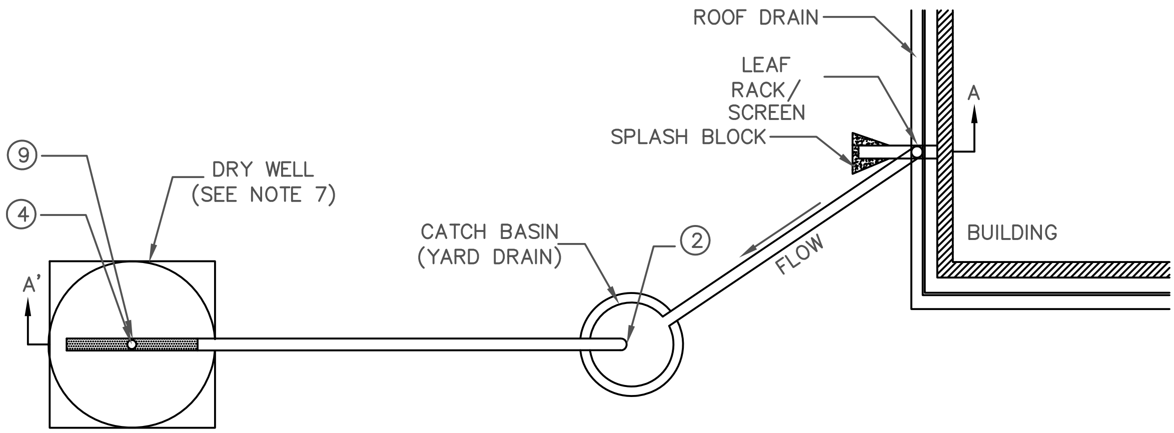
Photo Credits: 1. K&A Enterprises; 2. Canale Landscaping

Application

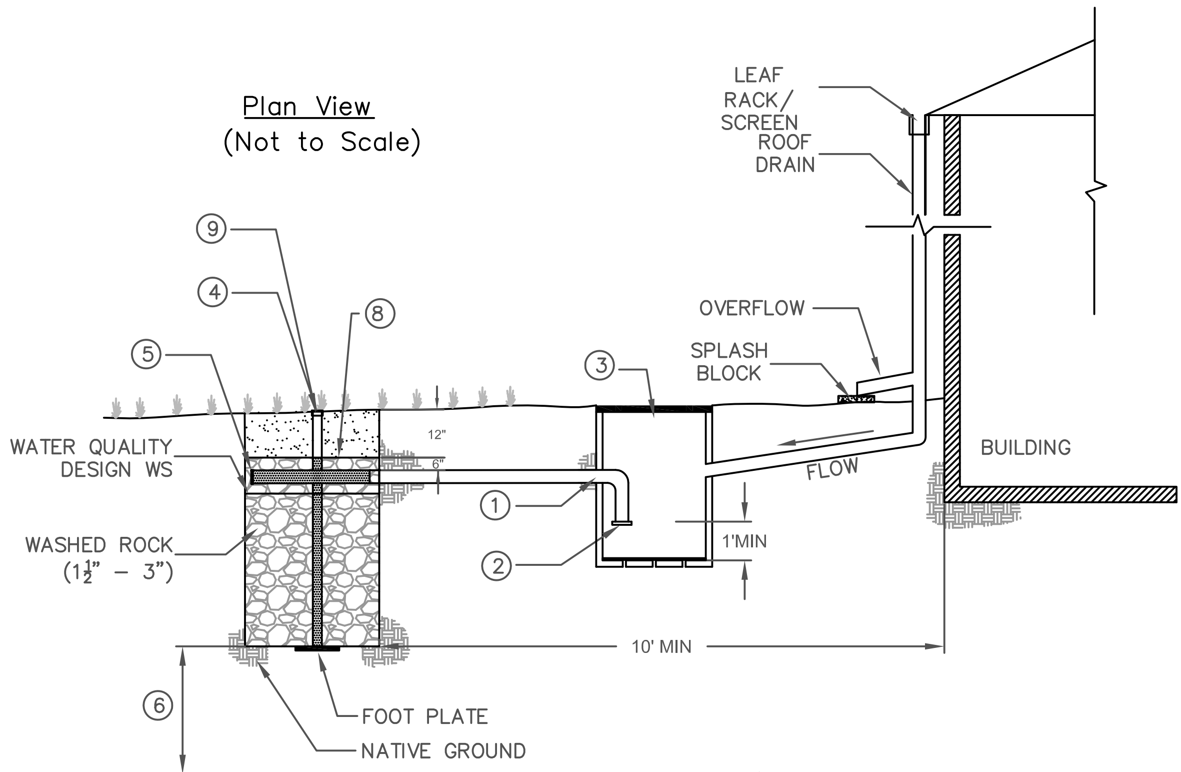
- Infiltration of roof runoff

Preventative Maintenance

- Remove trash, debris, and sediment at inlet and outlets
- Wet weather inspection to ensure drain time
- Inspect for mosquito breeding



Plan View
(Not to Scale)



Section A – A'
(Not to Scale)

NOTES:

1 MINIMUM 4" – 6" DIAMETER PVC PIPE. INSTALL AT FLAT SLOPE.

2 INSTALL FINE MESH SCREEN AT INLET TO DRY WELL. SET INLET ELEVATION AT 1' MINIMUM ABOVE CATCH BASIN BOTTOM.

3 CATCH BASIN (YARD DRAIN) INSTALLED WITH A SOLID LID FLUSH WITH GROUND SURFACE.

4 4–6" VERTICAL PERFORATED PVC INSPECTION WELL WITH SCREW LID (NUT DOWN) FLUSH WITH GROUND SURFACE.

5 CAP END OF 4–6" HORZONTAL PERFORATED PVC DISPERSION PIPE.

6 MINIMUM 10' ABOVE SEASONAL HIGH GROUNDWATER TABLE AND 3' ABOVE BEDROCK.

7 DRY WELL CONFIGURATION MAY VARY (E.G. PRE–FAB MAY BE CIRCULAR).

8 CHOKING STONE LAYER SHALL BE PLACED ON TOP OF THE DRY WELL TO SEPARATE IT FROM THE TOPSOIL AND PREVENT CLOGGING.

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Figure 6-5: Drywell

Limitations

The following limitations shall be considered before choosing to use a dry well:

- Native soil infiltration rate – soil permeability at the infiltration basin location must be at least 0.5 inches per hour.
- Depth to groundwater, bedrock, or low permeability soil layer – 5 feet vertical separation is required between the bottom of the infiltration basin and the seasonal high groundwater level or mounded groundwater level, bedrock, or other barrier to infiltration to ensure that the facility will completely drain between storms and that infiltrating water will receive adequate treatment though the soils before it reaches the groundwater.
- Slope stability - infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.
- Setbacks - a minimum setback (100 feet or more) must be provided between infiltration BMPs and potable wells, non-potable wells, drain fields, and springs. Infiltration BMPs must be setback from building foundations at least eight feet or have an alternative setback established by the geotechnical expert for the project.
- Groundwater contamination - the application of infiltration BMPs should include significant pretreatment in an area identified as an unconfined aquifer, to ensure groundwater is protected from pollutants of concern.
- Contaminated soils or groundwater plumes - infiltration BMPs are not allowed at locations with contaminated soils or groundwater where the pollutants could be mobilized or exacerbated by infiltration, unless a site-specific analysis determines the infiltration would be beneficial.
- High pollutant land uses - infiltration BMPs should not be placed in high-risk areas such as at or near service/gas stations, truck stops, and heavy industrial sites due to groundwater contamination risk unless a site-specific evaluation demonstrates that sufficient pretreatment is provided to address pollutants of concern, high risks areas are isolated from stormwater runoff, or infiltration areas have little chance of spill migration.
- High sediment loading rates – infiltration BMPs may clog quickly if sediment loads are high (e.g., unstabilized site) or if flows are not adequately pretreated.
- Dry wells cannot receive untreated stormwater runoff, except rooftop runoff. Pretreatment of runoff from other surfaces is necessary to prevent premature failure that results from clogging with fine sediment, and to prevent potential groundwater contamination due to nutrients, salts, and hydrocarbons.

- Infiltration structures cannot be used to treat runoff from portions of the site that are not stabilized.
- Rehabilitation of failed dry wells requires complete reconstruction.

Design Criteria

The main challenge associated with drywells, as with infiltration trenches, is the prevention of system clogging and subsequent infiltration inhibition. Drywells should be designed according to the requirements listed in Table 6-10 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-10: Infiltration BMP Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume (SQDV)	acre-feet	See Section 2 and Appendix E for calculating SQDV.
Design drawdown time	hour	12
Pretreatment	-	BIO-3: Vegetated Swale, BIO-4: Filter Strip, proprietary device, or equivalent.
Design percolation rate (k_{design})	in/hr	Shall be corrected for testing method, potential for clogging and compaction over time, and facility geometry.
Maximum depth of facility (d_{max})	feet	Defined by the design infiltration rate and the design drawdown time (includes depth of media).
Surface area of facility (A)	ft^2	Based on depth of dry well media.
Facility geometry	-	Geometry varies; max 10 feet deep; flat bottom slope.
Filter media diameter	inches	1.5 – 3 (gravel); prefabricated media may also be used
Overflow device	-	Required if system is on-line

Geotechnical Considerations

An extensive geotechnical site investigation must be undertaken early in the site planning process to verify site suitability for the installation of infiltration facilities, due to the potential to contaminate groundwater, cause slope instability, impact surrounding structures, and have insufficient infiltration capacity. Soil infiltration rates and the water table depth should be evaluated to ensure that conditions are satisfactory for proper operation of an infiltration facility. See Appendix C for guidance on infiltration testing.

The project designer must demonstrate through infiltration testing, soil logs, and the written opinion of a licensed civil engineer that sufficiently permeable soils exist on site to allow the construction of a properly functioning infiltration facility.

1) Infiltration facilities require a minimum soil infiltration rate of 0.5 inches/hour. If infiltration rates exceed 2.4 inches/hour, then the runoff should be fully-treated in an upstream BMP prior to infiltration to protect groundwater quality. Pretreatment for coarse sediment removal is required in all instances.

2) Groundwater separation must be at least 5 feet from the basin bottom to the measured season high groundwater elevation or estimated high groundwater mounding elevation. Measurements of groundwater levels must be made during the time when water level is expected to be at a maximum (i.e., toward the end of the wet season).

3) Sites with a slope greater than 25% (4:1) should be excluded. A geotechnical analysis and report addressing slope stability are required if located on slopes greater than 15%.

Soil Assessment and Site Geotechnical Investigation Reports

The soil assessment report should:

- State whether the site is suitable for the proposed drywell;

- Recommend a design infiltration rate (see the Step 2 of sizing methodology section, "Determine the design percolation rate," in the INF-1: Infiltration Basin fact sheet above);

- Identify the seasonal high depth to groundwater table surface elevation;

- Provide a good understanding of how the stormwater runoff will move in the soil (horizontally or vertically) and if there are any geological conditions that could inhibit the movement of water; and

- If a geotechnical investigation and report are required, the report should:

  - Provide a written opinion by a professional civil engineer describing whether the drywell will compromise slope stability; and

  - Identify potential impacts to nearby structural foundations.

Setbacks

1) Infiltration facilities shall be setback a minimum of 100 feet from proposed or existing potable wells, non-potable wells, septic drain fields, and springs.

2) Infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.

3) Infiltration BMPs must be setback from building foundations at least eight feet or have an alternative setback established by the geotechnical expert for the project.

Pretreatment

- A removable filter with a screened bottom should be installed in the roof leader below the surcharge pipe in order to screen out leaves and other debris.

- Though roofs are generally not a significant source of runoff pollution, they can still be source of particulates and organic matter. Measures such as roof gutter guards, roof leader clean-out with sump, or an intermediate sump box can provide pretreatment for dry wells by minimizing the amount of sediment and other particulates that may enter it.

Sizing Criteria

See Sizing Criteria section in the INF-1: Infiltration Basin fact sheet.

Geometry and Sizing

1) Dry well configurations vary, but generally they have length and width dimensions closer to square than infiltration trenches. Pre-fabricated dry-wells are often circular. The surface area of the dry well must be large enough to infiltrate the storage volume in 12 hours based on the maximum depth allowable (d_{max}).

2) The filter bed media layers are the same as for infiltration trenches unless prefabricated dry wells and/or media are used. The porosity of gravel media systems is generally 30 to 40% and is 80 to 95% for prefabricated media systems.

3) If a dry well receives runoff from an underground pipe (i.e., runoff does not enter the top of the dry well from the ground surface), a fine mesh screen should be installed at the inlet. The inlet elevation should be 18 inches below the ground surface (i.e., below 12 inches of surface soil and 6 inches of dry well media).

4) An observation well should be installed to check for water levels, drawdown time, and evidence of clogging. A typical observation well consists of a slotted PVC well screen, 4 to 6 inches in diameter, capped with a lockable, above-ground lid.

Drainage

1) The bottom of infiltration bed must be native soil, over-excavated to at least one foot in depth and replaced uniformly without compaction. Amending the excavated soil with 2 to 4 inches (~15% to 30%) of coarse sand is recommended.

2) The hydraulic conductivity of the subsurface layers should be sufficient to ensure a maximum 12 hr drawdown time. An observation well should be incorporated to allow observation of drain time.

Emergency Overflow

1) There must be an overflow route for stormwater flows that overtop the facility or in case the infiltration facility becomes clogged.

2) The overflow channel must be able to safely convey flows from the peak design storm to the downstream stormwater conveyance system or other acceptable discharge point.

Vegetation

1) Drywells should be kept free of vegetation.

2) Trees and other large vegetation should be planted away from drywells such that drip lines do not overhang infiltration beds.

Maintenance Access

1) The facility and outlet structures must all be safely accessible during wet and dry weather conditions.

2) Maintenance access is required.

3) If the drywell becomes plugged and fails, then access is needed to excavate the facility to remove and replace the top layer and the filter bed media of the structure. To prevent damage and compaction, access must be able to accommodate a backhoe working at “arms length”.

Construction Considerations

To preserve and avoid the loss of infiltration capacity, the following construction guidelines should be specified:

1) The entire area draining to the facility must be stabilized before construction begins. If this is impossible, a diversion berm should be placed around the perimeter of the infiltration site to prevent sediment entering during construction.

2) Drywells should not be hydraulically connected to the stormwater conveyance system until all contributing tributary areas are stabilized as shown on the Contract Plans and to the satisfaction of the Engineer. Drywells should not be used as sediment control facilities.

3) Compaction of the subgrade with heavy equipment should be minimized to the maximum extent possible. If the use of heavy equipment on the base of the facility

cannot be avoided, the infiltration capacity should be restored by tilling or aerating prior to placing the infiltrative bed.

4) The exposed soils should be inspected by a civil engineer after excavation to confirm that soil conditions are suitable.

Operations and Maintenance

Drywell maintenance should be performed frequently to ensure that water infiltrates into the subsurface completely within the recommended infiltration time (or drain time if a drywell receives runoff from an underground pipe) of 72 hours or less after a storm.

Maintenance and regular inspections are important for the proper function of drywells. A specific maintenance plan shall be developed specifically for each facility outlining the schedule and scope of maintenance operations, documentation, and reporting requirements.

INF-5: Permeable Pavement

Permeable pavements contain small voids that allow water to pass through to a stone base. They come in a variety of forms; they may be a modular paving system (concrete pavers, grass-pave, or gravel-pave) or a poured-in-place solution (porous concrete or permeable asphalt). All permeable pavements with a stone reservoir base treat stormwater and remove sediments and metals to some degree. While conventional pavement result in increased rates and volumes of surface runoff, porous pavements when properly constructed and maintained, allow some of the stormwater to percolate through the pavement and enter the soil below. This facilitates groundwater recharge while providing the structural and functional features needed for the roadway, parking lot, or sidewalk. The paving surface, subgrade, and installation requirements of permeable pavements are more complex than those for conventional asphalt or concrete surfaces. For porous pavements to function properly over an expected life span of 15 to 20 years, they must be properly sited and carefully designed and installed, as well as periodically maintained. Failure to protect paved areas from construction-related sediment loads can result in their premature clogging and failure. Note that the 2011 TGM does not provide specific instructions on how to design and construct pavement.

Permeable pavement applications

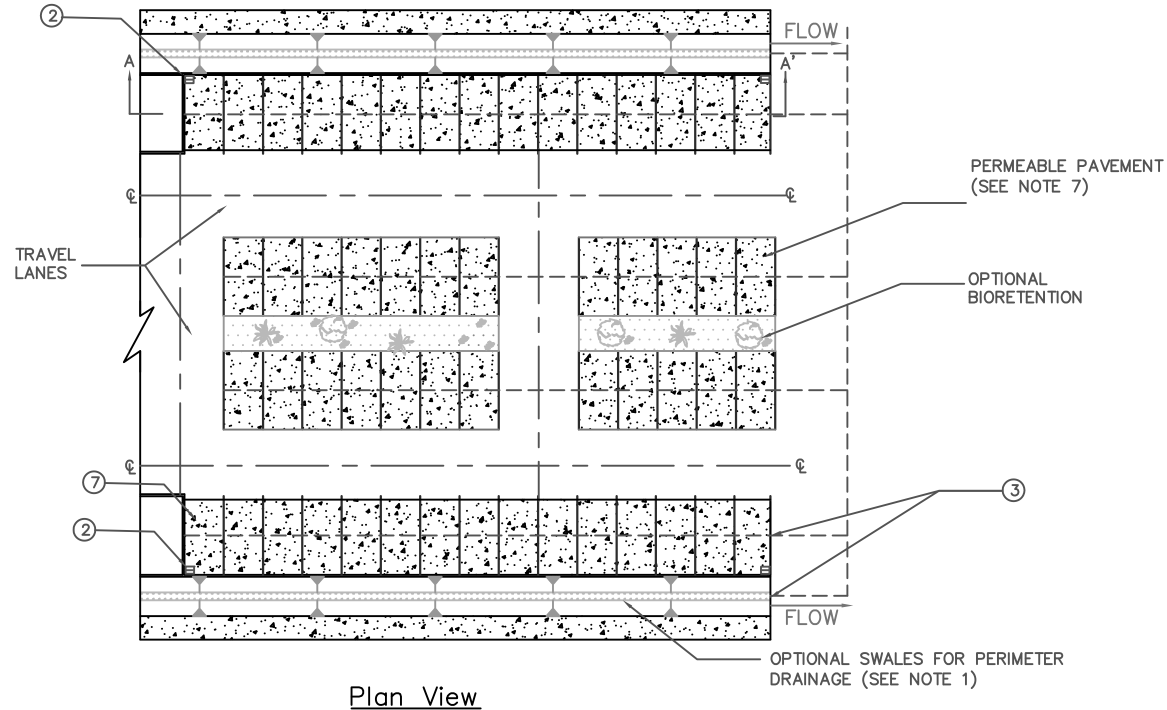
Photo Credits: 1. Geosyntec Consultants; 2. EPA Stormwater Management

Application

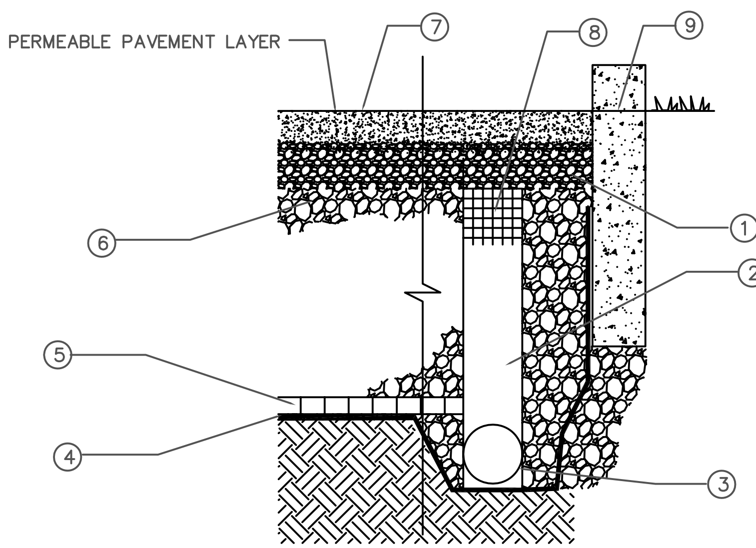
- Parking lots
- Driveways
- Sidewalks and walkways
- Outdoor athletic courts

Preventative Maintenance

- Trash removal
- Post-rain inspections
- Vacuum sweeping
- Vegetation inspection and removal



Plan View
(Not to Scale)



Section A – A'
(Not to Scale)

NOTES:

1. BEDDING COURSE SHALL BE 1½" TO 3" MIN THICKNESS (TYP NO. 8 AGGREGATE).
2. OPTIONAL OVERFLOW PIPE(S) SHALL BE PROVIDED IF OVERFLOWS ARE NOT MANAGED VIA PERIMETER DRAINAGE TO SWALES, BIORETENTION OR STORM WATER CONVEYANCE SYSTEM INLETS.
3. CONNECT OUTFALL PIPES TO DOWNSTREAM STORMWATER CONVEYANCE SYSTEM. OUTFALL PIPES SHALL BE SLOPED TOWARDS COLLECTION SYSTEM.
4. SOIL SUBGRADE SHALL HAVE ZERO SLOPE.
5. INSTALL GEOTEXTILE OR CHOKING LAYER ON BOTTOM & SIDES OF OPEN-GRADED BASE FOR FULL AND PARTIAL INFILTRATION, OR AN IMPERMEABLE LINER FOR NO INFILTRATION.
6. OPEN-GRADED BASE. THICKNESS AND GRADATION VARIES WITH DESIGN. TYP. NO. 57 AGGREGATE OR 4" THICK NO. 57 OVER NO. 2 STONE SUBASE. THICKNESS OF SUB-BASE VARIES WITH DESIGN.
7. PERMEABLE PAVEMENT INFILTRATIVE LAYER
8. OPTIONAL RIGID PLASTIC SCREEN FASTENED OVER OVERFLOW INLETS.
9. CURB/EDGE RESTRAINT WITH CUT-OUTS FOR OVERFLOW DRAINAGE TO PERIMETER BMPS, STORMWATER CONVEYANCE SYSTEM INLETS OR OPTIONAL OVERFLOW PIPES.
10. PARTIAL EXFILTRATION THROUGH THE SOIL. PERFORATED PIPES DRAIN EXCESS RUNOFF THAT CAN NOT BE ABSORBED BY SLOW-DRAINING SOIL.

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Figure 6-6: Permeable Pavement

Limitations

The following describes limitations for the use of permeable pavement.

- Native soil infiltration rate - permeability of soils at the BMP location must be at least 0.5 inches per hour.
- Depth to groundwater, bedrock, or low permeability soil layer – 5 feet vertical separation is required between the bottom of the infiltration trench and the seasonal high groundwater level or mounded groundwater level, bedrock, or other infiltration barrier to ensure that the facility will completely drain between storms and that infiltrating water will receive adequate treatment though the soils before it reaches the groundwater.
- Slope stability - infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.
- Setbacks - a minimum setback (100 feet or more) must be provided between infiltration BMPs and potable wells, non-potable wells, drain fields, and springs. Infiltration BMPs must be setback from building foundations at least eight feet or an alternative setback established by the geotechnical expert for the project.
- Groundwater contamination - the application of infiltration BMPs should include significant pretreatment in an area identified as an unconfined aquifer, to ensure groundwater is protected for pollutants of concern.
- Contaminated soils or groundwater plumes - infiltration BMPs are not allowed at locations with contaminated soils or groundwater where the pollutants could be mobilized or exacerbated by infiltration, unless a site-specific analysis determines the infiltration would be beneficial.
- High pollutant land uses - infiltration BMPs should not be placed in high-risk areas such as at or near a service/gas stations, truck stops, and heavy industrial sites due to the groundwater contamination risk unless a site-specific evaluation demonstrates that sufficient pretreatment is provided to address pollutants of concern, high risks areas are isolated from stormwater runoff, or infiltration areas that have little chance of spill migration.
- High sediment loading rates – infiltration BMPs may clog quickly if sediment loads are high (e.g., unstabilized site) or if flows are not adequately pretreated.
- Permeable pavement cannot receive untreated stormwater runoff from other surfaces. Pretreatment of run-on from other surfaces is necessary to prevent premature failure that results from clogging with fine sediment.

- Permeable pavement cannot be used to treat runoff from portions of the site that are not stabilized.

Design Criteria

Permeable pavement should be designed according to the requirements listed in Table 6-11 and outlined in the section below.

Table 6-11: Permeable Pavements Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater Quality Design Volume (SQDV)	acre-feet	See Section 2 and Appendix E for calculating SQDV.
Pretreatment	-	Runoff from pervious areas should be minimized but, if provided, BIO-3: Vegetated Swale or BIO-4: Filter Strip should be provided for all runoff from offsite sources that are not directly adjacent to the permeable pavement.
Drawdown time of gravel drainage layer	hrs	12 - 72
Porous Pavement Infill		ASTM C-33 sand or equivalent
Minimum depth to bedrock	ft	2 (without underdrains)
Minimum depth to seasonal high water table	ft	2 (with underdrains); 10 (without underdrains)
Infiltration rate of subsoil	in/hr	1.0 (minimum without an underdrain)
Overflow device	-	Required

Geotechnical Considerations

An extensive geotechnical site investigation must be undertaken early in the site planning process to verify site suitability for the installation of infiltration facilities, due to the potential to contaminate groundwater, cause slope instability, impact surrounding structures, and have insufficient infiltration capacity. Soil infiltration rates and the water table depth should be evaluated to ensure that conditions are satisfactory for proper operation of an infiltration facility. See Appendix C for guidance on infiltration testing.

The project designer must demonstrate through infiltration testing, soil logs, and the written opinion of a licensed civil engineer that sufficiently permeable soils exist onsite to allow the construction of a properly functioning infiltration facility.

1) Infiltration facilities require a minimum native soil infiltration rate of 0.5 inches/hour. If infiltration rates exceed 2.4 inches/hour, then the runoff should be fully treated in an upstream BMP prior to infiltration to protect groundwater quality.

Pretreatment for removing coarse sediment present in runoff from the tributary area is required in all instances.

2) Groundwater separation must be at least 5 feet from the basin bottom to the measured season high groundwater elevation or estimated high groundwater mounding elevation. Groundwater levels measurements must be made during the time when the water level is expected to be at a maximum (i.e., toward the end of the wet season).

3) Sites with a slope greater than 25% (4:1) should be excluded. A geotechnical analysis and report addressing slope stability are required if located on slopes greater than 15%.

Soil Assessment and Site Geotechnical Investigation Reports

The soil assessment report should:

- State whether the site is suitable for the proposed permeable pavement;
- Recommend a design infiltration rate (see the Step 2 of sizing methodology section, "Determine the design percolation rate," in the Infiltration Basin fact sheet above);
- Identify the seasonal high depth to groundwater table surface elevation;
- Provide a good understanding of how the stormwater runoff will move in the soil (horizontally or vertically) and if there are any geological conditions that could inhibit the movement of water; and
- If a geotechnical investigation and report are required, the report should:
  - Provide a written opinion by a professional civil engineer describing whether the infiltration trench will compromise slope stability; and
  - Identify potential impacts to nearby structural foundations.

Setbacks

1) Infiltration facilities shall be setback a minimum of 100 feet from proposed or existing potable wells, non-potable wells, septic drain fields, and springs.

2) Infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.

3) Infiltration BMPs must be setback from building foundations at least eight feet or have an alternative setback established by the geotechnical expert for the project.

Pretreatment

1) Depending on how and where permeable pavements will be used, pretreatment of the runoff entering the permeable pavement may be necessary. This is particularly important when the permeable pavement will be accepting run-on from pervious areas or areas that are not completely stabilized. If this is the case, then the run-on should be treated prior to contacting the permeable pavement. Without adequate pretreatment, the life of the permeable pavement may be significantly decreased.

2) If sheet flow is conveyed to the permeable pavement over stabilized grassed areas, the site must be graded in such a way that minimizes erosive conditions.

Sizing Criteria

Permeable pavement must be designed to meet Ventura County codes and/or applicable local permitting authority codes. These sizing criteria are meant to provide guidance for runoff volume storage only.

Step 1: Calculate the Design Volume

Infiltration facilities shall be sized to capture and infiltrate the SQDV volume (see Section 2 and Appendix E) with a 12 to 72 hour drawdown time (see Appendix D, Section D.2).

Step 2: Determine the Design Percolation Rate

The percolation rate will decline between maintenance cycles as the surface becomes occluded and particulates accumulate in the infiltration layer. Monitoring of actual facility performance has shown that the full-scale infiltration rate is far lower than the rate measured by small-scale testing. It is important that adequate conservatism is incorporated in the selection of design percolation rates. For infiltration trenches, the design percolation rate discussed here is the percolation rate of the underlying soils and not the percolation rate of the filter media bed (refer to the "Geometry and Sizing" section of INF-2 for the recommended composition of the filter media bed for infiltration trenches).

Considerations for Design Percolation Rate Corrections

Suitability assessment related considerations include (Table 6-12):

- Soil assessment methods – the site assessment extent (e.g., number of borings, test pits, etc.) and the measurement method used to estimate the short-term infiltration rate.

- Predominant soil texture/percent fines – soil texture and the percent of fines can greatly influence the potential for clogging.

- Site soil variability – site with spatially heterogeneous soils (vertically or horizontally) as determined from site investigations are more difficult to estimate average properties resulting in a higher level of uncertainty associated with initial estimates.

- Depth to seasonal high groundwater/impervious layer – groundwater mounding may become an issue during excessively wet conditions where shallow aquifers or shallow clay lenses are present.

Table 6-12: Suitability Assessment Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern	Medium Concern	Low Concern
Assessment methods	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates	Direct measurement of ≥ 20 percent of infiltration area with localized infiltration measurement methods (e.g., infiltrometer)	Direct measurement of ≥ 50 percent of infiltration area with localized infiltration measurement methods or Use of extensive test pit infiltration measurement methods
Ventura Hydrology Manual soil number (measured infiltration rate)	3 (f = 0.5 – 0.64)	4 or 5 (f = 0.65 – 0.91)	6 or 7 (f = 0.92 or higher)
Site soil variability	Highly variable soils indicated from site assessment or limited soil borings collected during site assessment	Soil borings/test pits indicate moderately homogeneous soils	Multiple soil borings/test pits indicate relatively homogeneous soils
Depth to groundwater/ impervious layer	<10 ft below facility bottom	10-30 ft below facility bottom	>30 below facility bottom

Localized infiltration testing refers to methods such as the double ring infiltrometer test (ASTM D3385-88) which measure infiltration rates over an area less than 10 sq-ft and do not attempt to account for soil heterogeneity. Extensive infiltration testing refers to methods that include excavating a significant portion of the proposed infiltration area, filling the excavation with water, and monitoring drawdown. In all cases, testing should be conducted in the area of the proposed BMP where, based on geotechnical data, soils appear least likely to support infiltration.

Design related considerations include (Table 6-13):

- Size of area tributary to facility – all things being equal, both physical and economic risk factors related to infiltration facilities increase with an increase in the tributary area served. Therefore facilities serving larger tributary areas should use more restrictive adjustment factors.

- Level of pretreatment/expected influent sediment loads – credit should be given for good pretreatment by allowing less restrictive factors to account for the reduced probability of clogging from high sediment loading. Also facilities designed to capture runoff from relatively clean surfaces such as rooftops are likely to see low sediment loads and therefore should be allowed to apply less restrictive safety factors.

- Redundancy – facilities that consist of multiple subsystems operating in parallel such that parts of the system remains functional when other parts fail and/or bypass should be rewarded for the built-in redundancy with less restrictive correction and safety factors. For example, if bypass flows would be at least partially treated in another BMP, the risk of discharging untreated runoff in the event of clogging the primary facility is reduced. A bioretention facility that overflows to a landscaped area is another example.

Compaction during construction – proper construction oversight is needed during construction to ensure that the bottom of the infiltration facility are not overly compacted. Facilities that do not commit to proper construction practices and oversight should have to use more restrictive correction and safety factors.

Table 6-13: Design Related Considerations for Infiltration Facility Safety Factors

Consideration	High Concern	Medium Concern	Low Concern
Tributary area size	Greater than 10 acres.	Greater than 2 acres but less than 10 acres.	2 acres or less.
Level of pre-treatment/ expected influent sediment loads	Pre-treatment from gross solids removal devices only, such as hydrodynamic separators, racks and screens AND tributary area includes landscaped areas, steep slopes, high traffic areas, or any other areas expected to produce high sediment, trash, or debris loads.	Good pre-treatment with BMPs that mitigate coarse sediments such as vegetated swales AND influent sediment loads from the tributary area are expected to be relatively low (e.g., low traffic, mild slopes, disconnected impervious areas, etc.).	Excellent pre-treatment with BMPs that mitigate fine sediments such as bioretention or media filtration OR sedimentation or facility only treats runoff from relatively clean surfaces, such as rooftops.
Redundancy of treatment	No redundancy in BMP treatment train.	Medium redundancy, other BMPs available in treatment train to maintain at least 50% of function of facility in event of failure.	High redundancy, multiple components capable of operating independently and in parallel, maintaining at least 90% of facility functionality in event of failure.
Compaction during construction	Construction of facility on a compacted site or elevated probability of unintended/ indirect compaction.	Medium probability of unintended/ indirect compaction.	Heavy equipment actively prohibited from infiltration areas during construction and low probability of unintended/ indirect compaction.

Adjust the measured short-term infiltration rate using a weighted average of several safety factors, using the worksheet shown in Table 6-14 below. The design percolation rate would be determined as follows:

- For each consideration shown in Table 6-12 and Table 6-13 above, determine whether the consideration is a high, medium, or low concern.
- For all high concerns assign a factor value of 3, for medium concerns assign a factor value of 2, and for low concerns assign a factor value of 1.
- Multiply each of the factors by the corresponding weight to get a product.

- Sum the products within each factor category to obtain a safety factor for each.
- Multiply the two safety factors together to get the final combined safety factor. If the combined safety factor is less than 2, then use 2 as the safety factor.
- Divide the measured short term infiltration rate by the combined safety factor to obtain the adjusted design percolation rate for use in sizing the infiltration facility.

Table 6-14: Infiltration Facility Safety Factor Determination Worksheet

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) p = w x v
A	Suitability Assessment	Soil assessment methods	0.25		
		Predominant soil texture	0.25		
		Site soil variability	0.25		
		Depth to groundwater / impervious layer	0.25		
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			
B	Design	Tributary area size	0.25		
		Level of pre-treatment/ expected sediment loads	0.25		
		Redundancy	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \Sigma p$			
Combined Safety Factor = S_A x S_B					

Note: The minimum combined adjustment factor shall not be less than 2.0 and the maximum combined adjustment factor shall not exceed 9.

Step 3: Determine the Gravel Drainage Layer Depth

Permeable pavement (including the base layers) should be designed to drain in less than 72 hours. The basis for this is that soils must be allowed to dry out periodically in order to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates, maintain adequate sub soil oxygen levels for healthy soil biota, and to provide proper soil conditions for biodegradation and retention of pollutants.

1) Calculate the maximum depth of runoff (d_{max}) that can be infiltrated within the drawdown time:

$$d_{\max} = \frac{P_{design} \bullet t}{12} \quad \text{(Equation 6-11)}$$

Where:

d_{max} = maximum depth that can be infiltrated (ft)

P_{design} = design percolation rate of underlying soils (in/hr) (see Step 2, above)

t = drawdown time (12-72 hours) (hr)

2) Select the gravel drainage layer depth, (l), such that:

$$d_{\max} \geq n \times l \qquad \text{(Equation 6-12)}$$

Where:

d_{max} = maximum depth that can be infiltrated (ft) (see 1) above)

n = gravel drainage layer porosity(unitless)(generally about 40% or 0.40 for gravel)

l = gravel drainage layer depth (ft)

Step 4: Determine infiltrating surface area

3) Calculate infiltrating surface area for permeable pavement (A):

$$A = \frac{SQDV}{\frac{TP_{design}}{12} + nl} \qquad \text{(Equation 6-13)}$$

Where:

P_{design} = design percolation rate of underlying soils (in/hr) (see Step 2, above)

n = gravel drainage layer porosity(unitless)[about 40% or 0.40 for gravel]

l = depth of gravel drainage layer (ft)

T = time to fill the gravel drainage layer with water (use 2 hours for most designs) (hr)

Geometry and Size

1) Permeable pavement shall be sized to capture and treat the stormwater quality design volume (SQDV).

2) Pavement design options include:

a. Full or partial infiltration – A design for full infiltration uses an open graded base for maximum infiltration and storage of stormwater. The water infiltrates directly into the base and through the soil. Pipes may provide drainage in overflow conditions. Partial infiltration does not rely completely on infiltration through the soil to dispose all of the captured runoff. Some of the water may infiltrate into the soil and the remainder drained by pipes.

b. No infiltration – No infiltration is desirable when the soil has low permeability and low strength, or there are other site limitations. An underdrain should be provided if the depth to bedrock is less than 2 feet or the depth to the water table is less than 10 feet. By storing water for a time in the base and then slowly releasing it through pipes, the design behaves like an underground detention pond. In other cases, the soil of the sub-base may be compacted and stabilized to render improved support for vehicular loads. This practice reduces infiltration into the soil to nearly zero. The "no infiltration" option requires the use of geotextile and bedding between the pavement and the open graded base.

3) If permeable pavement is located on a site with a slope greater than 2%, the permeable pavement area should be terraced to prevent lateral flow through the subsurface. Permeable pavement cannot be located on a site with a slope greater than 5%.

4) Porous pavement systems generally consist of at least four different layers of material:

a. The top or wearing layer consists of either asphalt or concrete with a greater than normal percentage of voids (typically 12 to 20 percent in the case of asphalt). The wearing layer may also be comprised of lattice-type pavers (either hollow concrete blocks or paving stones made from solid conventional concrete or stone), which are set in a bedding material (sand, pea-sized gravel or turf grass).

b. Below the wearing layer, a stone reservoir layer or a thick layer of aggregate (e.g., 2 inch stone) provides the bulk of the water storage capacity for a porous pavement system. In the pavement design, it is important to ensure that this reservoir layer retains its load bearing capacity under saturated conditions, because it may take several days for complete drainage to occur.

c. Typically, porous pavement designs include two (or more) transition layers that can be constructed from 1 to 2 inch diameter stone. One transition layer separates the top wearing layer from the underlying stone reservoir layer. Another transition layer is used to separate the stone reservoir from the undisturbed subgrade soil. Some designs also add a geotextile layer to this bottom layer or some combination of stones and geotextiles.

d. Porous asphalt pavement, for example, consists of open grade asphalt mixture ranging in depth from 2 to 4 inches with 16 percent voids. The thickness selected depends on bearing strength and pavement design requirements. This layer sits on a 2 to 4 inch transition layer located over a stone reservoir. The bottom layer completes the transition to the underlying undisturbed soil using a combination transition/filter fabric layer.

e. The depth of each layer should be determined by a licensed civil engineer based on analyses of the hydrology, hydraulics, and structural requirements of the site.

5) Modular paving stones are also used to create porous pavements. These pavements can be constructed in situ by pouring concrete into special frames or by using preformed blocks. The top layer of these porous pavements consists of conventional concrete, with the intervening void areas filled with either turf or sand. A transition or bedding layer is used to make the transition to the reservoir layer. These lattice-type pavers or hollow concrete blocks are often used in conjunction with turf grasses and are used in low-traffic parking lots, lanes, or driveways. Porous pavements using paving stones have similar construction, but can be designed to have a much higher load bearing capacity, and therefore have more widespread applicability. Construction guidelines and design specifications are available from the manufacturers of these products.

6) Permeable pavement (including the base layers) should be designed to drain in less than 72 hours. The basis for this is that soils must be allowed to dry out periodically in order to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates, maintain adequate subsoil oxygen levels for healthy soil biota, and to provide proper soil conditions for biodegradation and retention of pollutants.

7) The percolation rate will decline as the surface becomes occluded and particulates accumulate in the infiltration layer. It is important that adequate conservatism is incorporated in the selection of design percolation rates.

Overflow

An overflow mechanism is required. Two options are provided:

Option 1: Perimeter control

Flows in excess of the design capacity of the permeable pavement system will require an overflow system connected to a downstream conveyance or other stormwater runoff BMP. In addition, if the pavement becomes clogged and infiltration decreases to the point that there is ponding, runoff will migrate off of the pavement via overland flow instead of infiltrating into the subsurface gravel layer. There are several options for handling overflow using perimeter controls such as:

1) Perimeter vegetated swale.
2) Perimeter bioretention.
3) Storm drain inlets.
4) Rock filled trench that funnels flow around pavement and into the subsurface gravel layer.

Option 2: Overflow pipe(s)

1) A vertical pipe should be connected to the underdrain.
2) The diameter, location, and quantity may vary with design and should be determined by a licensed civil engineer.
3) The pipe should be located away from vehicular traffic.
4) The piping system may incorporate an observational and/or cleanout well.
5) The top of the overflow pipe should be covered with a screen fastened over the overflow inlet.

Construction Considerations

1) Permeable pavement should be laid close to level and the bottom of the base layers must be level to ensure uniform infiltration.
2) Permeable pavement surfaces should not be used to store site materials, unless the surface is well protected from accidental spillage or other contamination.
3) To prevent/minimize soil compaction in the area of the permeable pavement installation, use light equipment with tracks or oversized tires.
4) Divert stormwater from the area as needed (before and during installation).
5) The pavement should be the last installation done at a development site. Landscaping should be completed and adjacent areas stabilized, before pavement installation to minimize the risk of clogging.
6) Vehicular traffic should be prohibited for at least 2 days after installation.

Operations and Maintenance

Permeable pavement mainly requires vacuuming and management of adjacent areas to limit sediment contamination and prevent clogging by fine sediment particles. Therefore, little special training is needed for maintenance crews. The following maintenance concerns and maintenance activities shall be considered and provided:

1) Trash tends to accumulate in paved areas, particularly in parking lots and along roadways. The need for litter removal should be determined through periodic inspection.

2) Regularly (e.g., monthly for a few months after initial installation, then quarterly) inspect pavement for pools of standing water after rain events, this could indicate surface clogging.

3) Actively (3 to 4 times per year, or more frequently depending onsite conditions) vacuum sweep the pavement to reduce the risk of clogging by frequently removing fine sediments before they can clog the pavement and subsurface layers. This also helps to prolong the functional period of the pavement.

4) Inspect for vegetation growth on pavement and remove when present.

5) Inspect for missing sand/gravel in spaces between pavers and replace as needed.

6) Activities that lead to ruts or depressions on the surface should be prevented or the integrity of the pavement should be restored by patching or repaving. Examples are vehicle tracks and utility maintenance.

7) Spot clogging of porous concrete may be remedied by drilling 0.5 inch holes every few feet in the concrete.

8) Interlocking pavers that are damaged should be replaced.

9) Maintain landscaped areas and reseed bare areas.

INF-6: Proprietary Infiltration

A number of vendors offer proprietary infiltration products that allow for similar or enhanced rates of infiltration and subsurface storage while offering durable prefrabricated structures. There are many varieties of proprietary infiltration BMPs.

Proprietary Infiltration BMPs

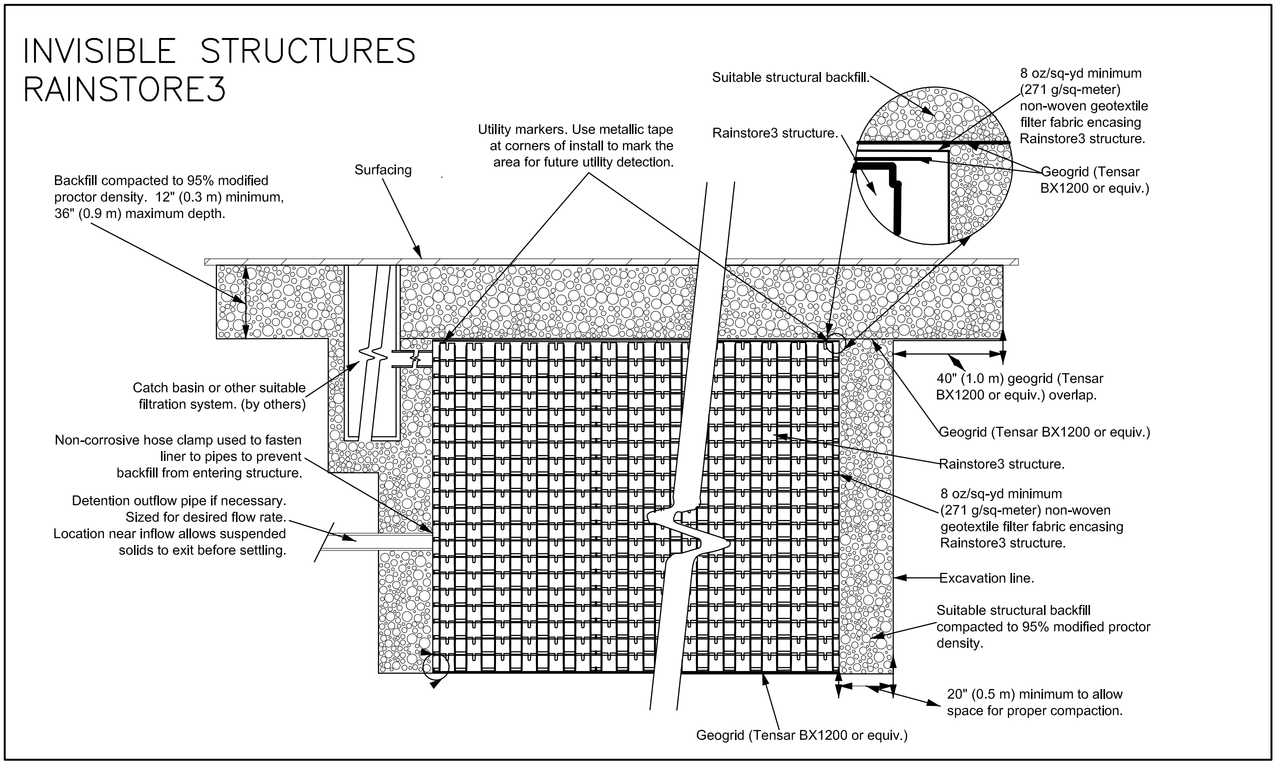
Photo Credits: 1. & 2. Contech Stormwater Solutions, Inc.

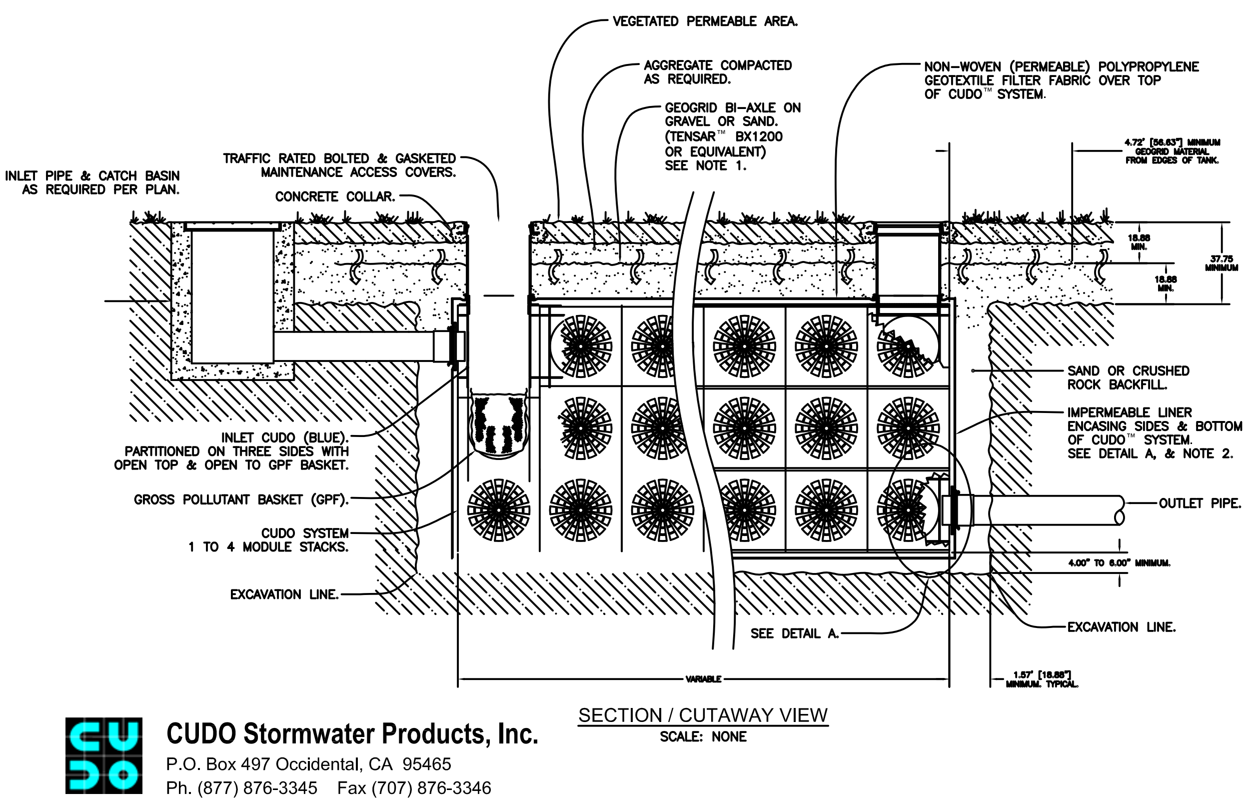
Application

- Mixed-use and commercial
- Roads and parking lots
- Parks and open spaces
- Single and multi-family residential

Routine Maintenance

- Removal trash, debris, and sediment at inlet and outlets
- Wet weather inspection to ensure drain time
- Inspect for mosquito breeding





Geosyntec consultants

Figure 6-7: Proprietary Infiltration BMPs

Limitations

The following limitations shall be considered before choosing to use an infiltration BMP:

- Native soil infiltration rate - soil permeability of the infiltration basin location must be at least 0.5 inches per hour.
- Depth to groundwater, bedrock, or low permeability soil layer – 5 feet vertical separation is required between the bottom of the infiltration basin and the seasonal high groundwater level or mounded groundwater level, bedrock, or other barrier to infiltration to ensure that the facility will completely drain between storms and that infiltrating water will receive adequate treatment though the soils before it reaches the groundwater.
- Slope stability - infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.
- Setbacks - a minimum setback (100 feet or more) must be provided between infiltration BMPs and potable wells, non-potable wells, drain fields and springs. Infiltration BMPs must be setback from building foundations at least eight feet or have an alternative setback established by the geotechnical expert for the project.
- Groundwater contamination - the application of infiltration BMPs should include significant pretreatment in an area identified as an unconfined aquifer, to ensure groundwater is protected for pollutants of concern.
- Contaminated soils or groundwater plumes - infiltration BMPs are not allowed at locations with contaminated soils or groundwater where the pollutants could be mobilized or exacerbated by infiltration, unless a site-specific analysis determines the infiltration would be beneficial.
- High pollutant land uses - infiltration BMPs should not be placed in high-risk areas such as at or near service/gas stations, truck stops, and heavy industrial sites due to the groundwater contamination risk unless a site-specific evaluation demonstrates that sufficient pretreatment is provided to address pollutants of concern, high risks areas are isolated from stormwater runoff, or infiltration areas have little chance of spill migration
- High sediment loading rates – infiltration BMPs may clog quickly if sediment loads are high (e.g., unstabilized site) or if flows are not adequately pretreated.

Table 6-15: Proprietary Infiltration Manufacturer Websites

Device	Manufacturer	Website
A-2000™	Contech® Construction Products Inc.	www.contech-cpi.com/stormwater/13
ChamberMaxx™	Contech® Construction Products Inc.	www.contech-cpi.com/stormwater/13
CON/SPAN Vaults™	Contech® Construction Products Inc.	www.contech-cpi.com/stormwater/13
CON/Storm™	Contech® Construction Products Inc.	www.contech-cpi.com/stormwater/13
Perforated Corrugated Metal Pipe (CMP)	Contech® Construction Products Inc.	www.contech-cpi.com/stormwater/13
Drywell StormFilter	Contech® Construction Products Inc.	www.contech-cpi.com/stormwater/13
CUDO® Water Storage System	KriStar Enterprises Inc.	www.kristar.com
D-Raintank® Matrix Tank Modules	Atlantis®	www.atlantis-america.com
EcoRain™ Modular Rain Tank	EcoRain Systems Inc.	www.ecorain.com
Landmax®	Hancor®	www.hancor.com
Landsaver™	Hancor®	www.hancor.com
Precast Concrete Dry Well	Jensen Precast®	www.jensenprecast.com
Rainstore^3	Invisible Structures Inc.	www.invisiblestructures.com
StormChambers™	Hydrologic Solutions, Inc.	www.hydrologicsolutions.com
Stormtech® SC-740 and SC-310 Chambers	StormTech LLC	www.stormtech.com
StormTrap®	StormTrap	www.stormtrap.com
Triton Chambers™	Triton Stormwater Solutions	www.tritonsws.com

Geotechnical Considerations

An extensive geotechnical site investigation must be undertaken early in the site planning process to verify site suitability for the installation of infiltration facilities, due to the potential to contaminate groundwater, cause slope instability, impact surrounding structures, and have insufficient infiltration capacity. Soil infiltration rates and the water table depth should be evaluated to ensure that conditions are satisfactory for proper operation of an infiltration facility. See Appendix C for guidance on infiltration testing.

The project designer must demonstrate through infiltration testing, soil logs, and the written opinion of a licensed civil engineer that sufficiently permeable soils exist onsite to allow the construction of a properly functioning infiltration facility.

1) Infiltration facilities require a minimum soil infiltration rate of 0.5 inches/hour. If infiltration rates exceed 2.4 inches/hour such that pollutant removal may not be adequate to protect groundwater quality, then the runoff should be fully treated in an upstream BMP prior to infiltration to protect groundwater quality. Pretreatment for coarse sediment removal is required in all instances.

2) Groundwater separation must be at least 5 feet from the basin bottom to the measured season high groundwater elevation or estimated high groundwater mounding elevation. Measurements of groundwater levels must be made during the time when water level is expected to be at a maximum (i.e., toward the end of the wet season).

3) Sites with a slope greater than 25% (4:1) should be excluded. A geotechnical analysis and report addressing slope stability are required if located on slopes greater than 15%.

Soil Assessment and Site Geotechnical Investigation Reports

The soil assessment report should:

- State whether the site is suitable for the proposed proprietary infiltration BMP.;
- Recommend a design infiltration rate (see the Step 2 of sizing methodology section, "Determine the design percolation rate," in the Infiltration Basin fact sheet above);
- Identify the seasonal high depth to groundwater table surface elevation;
- Provide a good understanding of how the stormwater runoff will move in the soil (horizontally or vertically) and if there are any geological conditions that could inhibit the movement of water; and
- If a geotechnical investigation and report are required, the report should:
  - Provide a written opinion by a professional civil engineer describing whether the infiltration trench will compromise slope stability; and
  - Identify potential impacts to nearby structural foundations.

Setbacks

1) Infiltration facilities shall be setback a minimum of 100 feet from proposed or existing potable wells, non-potable wells, septic drain fields, and springs.

2) Infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.

3) Infiltration BMPs must be setback from building foundations at least eight feet or have an alternative setback established by the geotechnical expert for the project.

Pretreatment

Pretreatment is required for proprietary infiltration BMPs in order to reduce the sediment load entering the facility and maintain the infiltration rate of the facility. Pretreatment refers to design features that provide settling of sediment particles before runoff reaches a management practice. This eases the long-term maintenance burden and likelihood of failure. Pretreatment is important for most stormwater treatment BMPs, but it is particularly important for infiltration BMPs. To ensure that pretreatment mechanisms are effective, designers should incorporate sediment reduction practices. Sediment reduction BMPs may include vegetated swales, vegetated filter strips, sedimentation basins, sedimentation manholes and hydrodynamic separation devices. The use of at least two pretreatment devices is highly recommended for infiltration BMPs.

Sizing

1) Proprietary infiltration BMPs shall be sized to capture and treat the stormwater quality design volume (SQDV). See Section 2 and Appendix E for calculating for further detail.

2) The percolation rate will decline as the surface becomes occluded and particulates accumulate in the infiltrative layer. It is important that adequate conservatism is incorporated in the selection of design percolation rates.

3) For the sizing guidelines, refer to the manufacturer's website.

Operations and Maintenance

See vendor's website for maintenance requirements.

INF-7: Bioinfiltration

Bioinfiltration facilities are designed for partial infiltration of runoff and partial biotreatment. These facilities are similar to bioretention devices with underdrains, but the underdrain is raised above the gravel sump to facilitate infiltration. These facilities can be used in areas where there are no hazards associated with infiltration, but infiltration of the full DCV may not be feasible due to low infiltration rates (Soil Type 3) or high depths of fill. These facilities may not result in retention of the DCV but they can be used to meet the MEP standards.

Bioretention in Parkway and parking lots

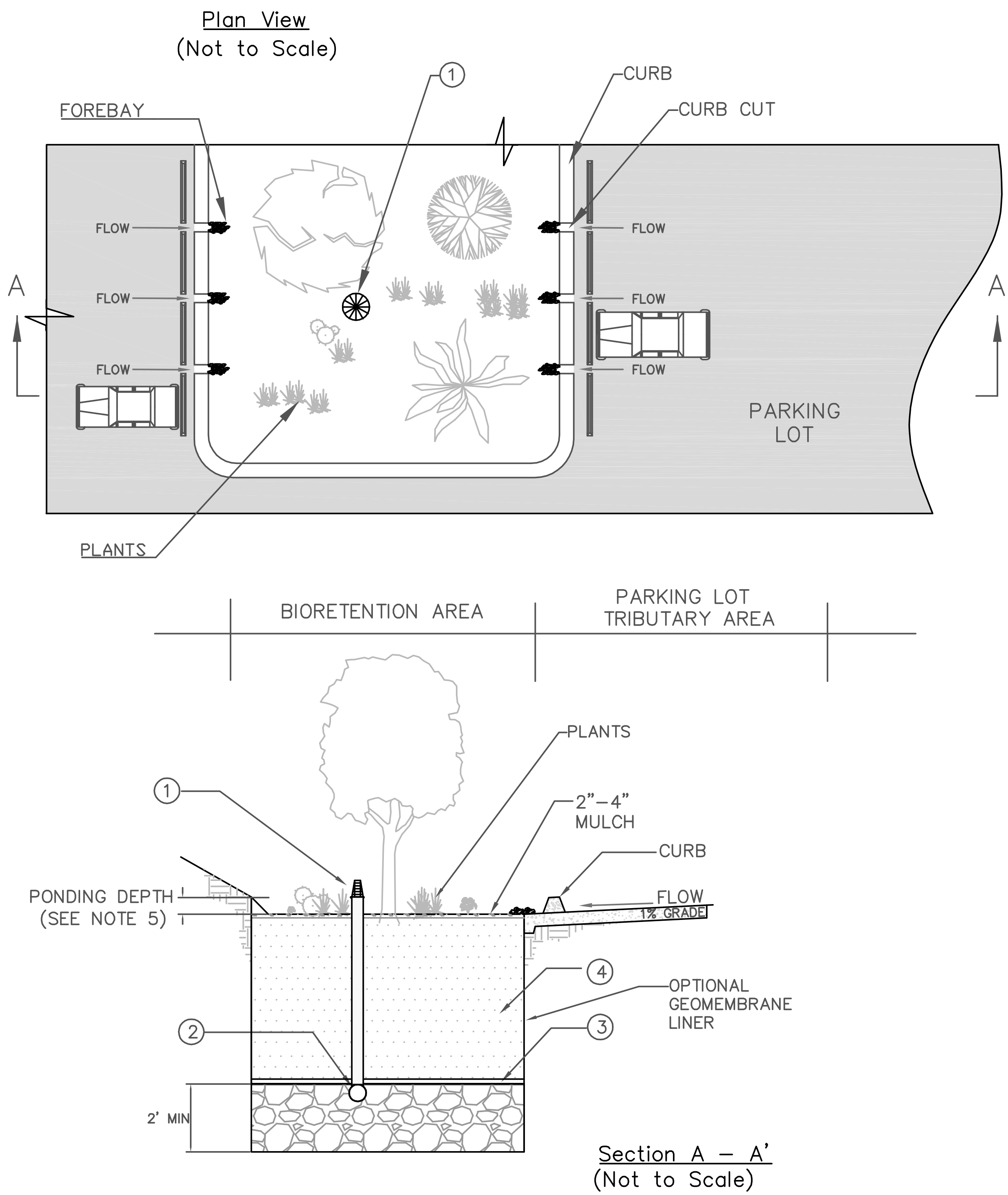
Photo Credits: Geosyntec Consultants

Application

- Commercial, residential, mixed use, institutional, and recreational uses
- Parking lot islands, traffic circles
- Road parkways & medians

Preventative Maintenance

- Repair small eroded areas
- Remove trash and debris and rake surface soils
- Remove accumulated fine sediments, dead leaves and trash
- Remove weeds and prune back excess plant growth
- Remove sediment and debris accumulation near inlet and outlet structures
- Periodically observe function under wet weather conditions



NOTES:

① OVERFLOW DEVICE: VERTICAL RISER OR EQUIVALENT.
② PERFORATED 6" MIN PVC PIPE UNDERDRAIN.
③ OPTIONAL CHOKING GRAVEL LAYER.
④ 2' MIN PLANTING MIX; 3' PREFERRED.
⑤ PONDING DEPTH 18" WITH FENCE; 6" WITHOUT FENCE.
⑥ 2' MIN GRAVEL LAYER DEPTH.

Geosyntec consultants

Figure 6-8: Bioinfiltration

Limitations

The following limitations should be considered before choosing to use bioinfiltration:

1) Native soil infiltration rate - soil permeability at the bioinfiltration location must be no less than 0.3 inches per hour.

2) Depth to groundwater, bedrock, or low permeability soil layer – 5 feet vertical separation is required between the bottom of the infiltration trench and the seasonal high groundwater level or mounded groundwater level, bedrock, or other barrier to infiltration to ensure that the facility will completely drain between storms and that infiltrating water will receive adequate treatment though the soils before it reaches the groundwater.

3) Slope stability - infiltration BMPs must be sited at least 50 feet away from slopes steeper than 15 percent or an alternative setback established by the geotechnical expert for the project.

4) Setbacks - a minimum setback (100 feet or more) must be provided between infiltration BMPs and potable wells, non-potable wells, drain fields, and springs. Infiltration BMPs must be setback from building foundations at least eight feet or have an alternative setback established by the geotechnical expert for the project.

5) Groundwater contamination - the application of infiltration BMPs should include significant pretreatment in an area identified as an unconfined aquifer to ensure groundwater is protected for pollutants of concern.

6) Contaminated soils or groundwater plumes - infiltration BMPs are not allowed at locations with contaminated soils or groundwater where the pollutants could be mobilized or exacerbated by infiltration, unless a site-specific analysis determines that infiltration would be beneficial.

7) High pollutant land uses - infiltration BMPs should not be placed in high-risk areas such as at or near service/gas stations, truck stops, and heavy industrial sites due to the groundwater contamination risk unless a site-specific evaluation demonstrates that sufficient pretreatment is provided to address pollutants of concern, high risks areas are isolated from stormwater runoff, or infiltration areas have little chance of spill migration.

8) High sediment loading rates – infiltration BMPs may clog quickly if sediment loads are high (e.g., unstabilized site) or if flows are not adequately pretreated.

9) Vertical relief and proximity to storm drain - site must have adequate relief between the land surface and storm drain to permit vertical percolation through the soil media and collection.

Design Criteria

Bioinfiltration should be designed according to the requirements listed in Table 6-16 and outlined in the section below.

Table 6-16: Bioretention Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume (SQDV)	acre-feet	See Section 2 and Appendix E for calculating SQDV.
Forebay	-	Forebay should be provided for all tributary surfaces that contain landscaped areas. Forebays should be designed to prevent standing water during dry weather and should be planted with a plant palette that is tolerant of wet conditions.
Maximum drawdown time of water ponded on surface	hours	48
Maximum drawdown time of surface ponding plus subsurface pores	hours	96 (72 preferred)
Maximum ponding depth	inches	18
Minimum thickness of amended soil	feet	2 (3 preferred)
Minimum thickness of stabilized mulch	inches	2 to 4
Planting mix composition	-	60 to 80% fine sand, 20 to 40% compost
Underdrain sizing	-	Underdrain should be installed below the choking stone; 6 inch minimum diameter; 0.5% minimum slope; slotted, polyvinyl chloride (PVC) pipe (PVC SDR 35 or approved equivalent); spacing shall be determined to provide capacity for maximum rate filtered through amended media
Minimum thickness of gravel layer	feet	2
Overflow device	-	Required

Sizing Criteria

Bioinfiltration facilities can be sized using one of two methods: a simple sizing method or a routing modeling method. With either method the SQDV volume must be completely infiltrated within 96 hours (including subsurface pore space), and surface ponding must be infiltrated within 48 hours. The simple sizing procedure is provided below. For the routing modeling method, refer to TCM-4 Sand Filters.

Step 1: Calculate the Design Volume

Bioinfiltration facilities shall be sized to capture and partially infiltrate and partially biotreat the SQDV volume (see Section 2.3 and Appendix E).

Step 2: Determine the Design Percolation Rate

The percolation rate through the BMP and to the subsurface will decline between maintenance cycles as the surface becomes occluded and particulates accumulate in the infiltration layer. Monitoring of actual facility performance has shown that the full-scale infiltration rate is far lower than the rate measured by small-scale testing. It is important that adequate conservatism is incorporated in the selection of design percolation rates. For bioinfiltration facilities, the design percolation rate discussed here is the adjusted percolation rate of the underlying soils and not the percolation rate of the filter media bed. The measured short-term infiltration rate should be adjusted using a factor of safety of 2.0.

Step 3: Calculate the surface area

Determine the size of the required infiltrating surface by assuming the SQDV will fill the available ponding depth plus the void spaces in the media, based on the computed porosity of the filter media and optional aggregate layer.

1) Determine the maximum depth of surface ponding that can be infiltrated within the required surface drain time (48 hr), (d_{max}), as follows:

$$d_{\max} = \frac{P_{design} \times t_{ponding}}{12\dfrac{in}{ft}} \qquad \text{(Equation 6-14)}$$

Where:

$t_{ponding}$	=	required drain time of surface ponding (48 hrs)
P_{design}	=	design percolation rate of underlying soils (in/hr) (see Step 2, above)

d_{max} = the maximum depth of surface ponding water that can be infiltrated within the required drain time (ft), calculated using Equation 6-14

2) Choose surface ponding depth (d_p) such that:

$$d_p \leq d_{\max} \qquad \text{(Equation 6-15)}$$

Where:

d_p = selected surface ponding depth (ft)

d_{max} = the maximum depth of water that can be infiltrated within the required drain time (ft)

Choose thickness(es) of amended media and aggregate layer(s) and calculate total effective storage depth of the bioinfiltration area ($d_{effective}$), as follows:

$$d_{effective} \leq (d_p + n^*_{media} l_{media} + n_{gravel} l_{gravel}) \qquad \text{(Equation 6-16)}$$

Where:

$d_{effective}$ = total equivalent depth of water stored in bioinfiltration area (ft), including surface ponding and volume available in pore spaces of media and gravel layers

d_p = surface ponding depth (ft), chosen using Equation 6=15

n^*_{media} = available porosity of amended soil media (ft/ft), approximately 0.25 ft/ft accounting for antecedent moisture conditions. This represents the volume of available pore space as a fraction of the total soil volume; sometimes has units of (ft^3/ft^3) or described as a percentage.

l_{media} = thickness of amended soil media layer (ft), minimum 2 ft

n_{gravel} = porosity of gravel layer (ft/ft), approximately 0.40 ft/ft

l_{gravel} = thickness of gravel layer (ft), minimum 2 ft

3) Check that entire effective depth (surface plus subsurface storage), $d_{effective}$, infiltrates in no greater than 96 hours as follows:

$$t_{total} = \frac{d_{effective}}{P_{design}} \times 12 \frac{in}{ft} \leq 96\ hr \qquad \text{(Equation 6-17)}$$

Where:

$d_{effective}$ = total equivalent depth of water stored in bioinfiltration area (ft), calculated using Equation 6-16

P_{design} = design percolation rate of underlying soils (in/hr) (see Step 2, above)

If $t_{total} > 96$ hrs, then reduce surface ponding depth and/or amended media thickness and/or gravel thickness and return to 1).

If $t_{total} \leq 96$ hrs, then proceed to 5).

4) Calculate required infiltrating surface area, (A_{req}):

$$A_{req} = \frac{SQDV}{d_{effective}} \qquad \text{(Equation 6-18)}$$

Where:

A_{req} = required infiltrating area (ft^2). Should be calculated at the contour corresponding to the mid ponding depth (i.e., $0.5 \times d_p$ from the bottom of the facility).

$SQDV$ = stormwater quality design volume (ft^3)

$d_{effective}$ = total equivalent depth of water stored in bioinfiltration area (ft), calculated using Equation 6-16

5) Calculate total footprint required by including a buffer for side slopes and freeboard; A_{req} is calculated at the contour corresponding to the mid ponding depth (i.e., $0.5 \times d_p$ from the bottom of the facility).

Geometry

1) Minimum planting soil depth should be 2 feet, although 3 feet is preferred.

   The intention is that the minimum planting soil depth should provide a beneficial root zone for the chosen plant palette and adequate water storage for the stormwater quality design volume. A deeper soil depth will provide a smaller surface area footprint.

2) Minimum gravel layer depth is 2 feet.

The intention is that the gravel sump provides partial retention of captured water.

3) Bioinfiltration should be designed to drain below the planting soil in less than 48 hours and completely drain from the gravel layer in 96 hours (both starting from the end of inflow).

   The intention is that soils must be allowed to dry out periodically in order to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates, maintain adequate soil oxygen levels for healthy soil biota and vegetation, and to provide proper soil conditions for biodegradation and retention of pollutants.

Flow Entrance and Energy Dissipation

The following types of flow entrance can be used for bioinfiltration cells:

1) Dispersed, low velocity flow across a landscape area. Dispersed flow may not be possible given space limitations or if the facility is controlling roadway or parking lot flows where curbs are mandatory.

2) Dispersed flow across pavement or gravel and past wheel stops for parking areas.

3) Curb cuts for roadside or parking lot areas: curb cuts should include rock or other erosion protection material in the channel entrance to dissipate energy. Flow entrance should drop 2 to 3 inches from curb line and it should provide a settling area and periodic sediment removal of coarse material before flow dissipates to the remainder of the cell.

4) Pipe flow entrance: Piped entrances, such as roof downspouts, should include rock, splash blocks, or other appropriate measures at the entrance to dissipate energy and disperse flows.

Woody plants (trees, shrubs, etc.) can restrict or concentrate flows and can be damaged by erosion around the root ball and should not be placed directly in the entrance flow path.

Underdrains

Underdrains should meet the following criteria:

1) 6-inch minimum diameter.

2) Underdrains should be made of slotted, polyvinyl chloride (PVC) pipe (PVC SDR 35 or approved equivalent). *The intention is that compared to round-hole perforated pipe, slotted underdrains provide greater intake capacity, clog resistant drainage, and reduced entrance velocity into the pipe, thereby reducing the chances of solids migration.*

3) Slotted pipe should have 2 to 4 rows of slots cut perpendicular to the axis of the pipe or at right angles to the pitch of corrugations. Slots should be 0.04 to 0.1 inches and should have a length of 1 to 1.25 inches. Slots should be longitudinally spaced such that the pipe has a minimum of one square inch of slot per lineal foot of pipe and should be placed with slots facing the bottom of the pipe.

4) Underdrains should be sloped at a minimum of 0.5%.

5) Rigid non-perforated observation pipes with a diameter equal to the underdrain diameter should be connected to the underdrain every 100 feet to provide a clean-out port as well as an observation well to monitor dewatering rates. The wells/cleanouts should be connected to the perforated underdrain with the appropriate manufactured connections. The wells/cleanouts should extend 6 inches above the top elevation of the bioinfiltration facility mulch, and should be capped with a lockable screw cap. The ends of the underdrain pipes not terminating in an observation well/cleanout should also be capped.

Gravel Layer

1) The following aggregate should be used for the gravel layer below the underdrain pipe. Place the underdrain below the choking stone, within the top 6 inches of the gravel layer.

Sieve size	Percent Passing
¾ inch	100
¼ inch	30-60
US No. 8	20-50
US No. 50	3-12
US No. 200	0-1

2) At the option of the designer/geotechnical engineer, a geotextile fabric may be placed between the planting media and the gravel layer. If a geotextile fabric is used, it should meet a minimum permittivity rate of 75 gal/min/ft^2, should not impede the infiltration rate of the soil medium, and should meet the following minimum materials requirements.

Geotextile Property	Value	Test Method
Trapezoidal Tear (lbs)	40 (min)	ASTM D4533
Permeability (cm/sec)	0.2 (min)	ASTM D4491
AOS (sieve size)	#60 - #70 (min)	ASTM D4751
Ultraviolet resistance	70% or greater	ASTM D4355

Preferably, aggregate (choking stone) should be used in place of filter fabric to reduce the potential for clogging. This aggregate layer should consist of 2 to 4 inches of washed sand underlain with 2 inches of choking stone (Typically #8 or #89 washed).

3) Bioinfiltration facilities have the added benefit of enhanced nitrogen removal due to the elevated underdrain. This allows for a fluctuating anaerobic/aerobic zone below the drain pipe. *The intention is that denitrification within the anaerobic/anoxic zone is facilitated by microbes using forms of nitrogen (NO_2 and NO_3) instead of oxygen for respiration.*

4) The underdrain should drain freely to an acceptable discharge point. The underdrain can be connected to a downstream open conveyance (vegetated swale), to another bioinfiltration cell as part of a connected treatment system, to a storm drain, daylight to a vegetated dispersion area using an effective flow dispersion device, or to a storage facility for harvesting.

Overflow

An overflow device is required at the 18-inch ponding depth. The following, or equivalent should be provided:

1) A vertical PVC pipe (SDR 35) to act as an overflow riser.

2) The overflow riser(s) should be 6 inches or greater in diameter, so it can be cleaned without damage to the pipe.

The inlet to the riser should be at the ponding depth (18 inches for fenced bioinfiltration areas and 6 inches for areas that are not fenced), and be capped with a spider cap to exclude floating mulch and debris. Spider caps should be screwed in or glued, i.e., not removable.

Hydraulic Restriction Layers

Infiltration pathways may need to be restricted due to the close proximity of roads, foundations, or other infrastructure. A geomembrane liner, or other equivalent water proofing, may be placed along the vertical walls to reduce lateral flows. This liner should have a minimum thickness of 30 mils.

Planting/Storage Media

1) The planting media placed in the cell should achieve a long-term, in-place infiltration rate of at least 1 inch per hour. Higher infiltration rates are permissible. If the design long-term, in-place infiltration rate of the soil exceeds 12 inches per hour, documentation should be provided to demonstrate that the media will adequately address pollutants of concern at a higher flowrate. Bioinfiltration soil shall also support vigorous plant growth.

2) Planting media should consist of 60 to 80% fine sand and 20 to 40% compost.

3) Sand should be free of wood, waste, coating such as clay, stone dust, carbonate, etc., or any other deleterious material. All aggregate passing the No. 200 sieve size should be non-plastic. Sand for bioinfiltration should be analyzed by an accredited lab using #200, #100, #40, #30, #16, #8, #4, and 3/8 sieves (ASTM D 422 or as approved by the local permitting authority) and meet the following gradation (Note: all sands complying with ASTM C33 for fine aggregate comply with the gradation requirements below):

Sieve Size (ASTM D422)	% Passing (by weight)	
	Minimum	Maximum
3/8 inch	100	100
#4	90	100
#8	70	100
#16	40	95
#30	15	70
#40	5	55
#100	0	15
#200	0	5

Note: the gradation of the sand component of the media is believed to be a major factor in the hydraulic conductivity of the media mix. If the desired hydraulic conductivity of the media cannot be achieved within the specified proportions of sand and compost (#2), then it may be necessary to utilize sand at the coarser end of the range specified in above (“minimum” column).

4) Compost should be a well decomposed, stable, weed free organic matter source derived from waste materials including yard debris, wood wastes, or other organic materials not including manure or biosolids meeting standards developed by the US Composting Council (USCC). The product shall be certified through the USCC Seal of Testing Assurance (STA) Program (a compost testing and information disclosure program). Compost quality should be verified via a lab analysis to be:

- Feedstock materials shall be specified and include one or more of the following: landscape/yard trimmings, grass clippings, food scraps, and agricultural crop residues.
- Organic matter: 35-75% dry weight basis.
- Carbon and Nitrogen Ratio: 15:1 < C:N < 25:1
- Maturity/Stability: shall have dark brown color and a soil-like odor. Compost exhibiting a sour or putrid smell, containing recognizable grass or leaves, or is hot (120 F) upon delivery or rewetting is not acceptable.

- Toxicity: any one of the following measures is sufficient to indicate non-toxicity:
  - NH4:NH3 < 3
  - Ammonium < 500 ppm, dry weight basis
  - Seed Germination > 80% of control
  - Plant trials > 80% of control
  - e. Solvita® > 5 index value
- Nutrient content:
  - Total Nitrogen content 0.9% or above preferred
  - Total Boron should be <80 ppm, soluble boron < 2.5 ppm
- Salinity: < 6.0 mmhos/cm
- pH between 6.5 and 8 (may vary with plant palette)

Compost for bioinfiltration should be analyzed by an accredited lab using #200, ¼ inch, ½ inch, and 1 inch sieves (ASTM D 422 or as approved by the local permitting authority) and meet the following gradation:

Sieve Size (ASTM D422)	% Passing (by weight)	
	Minimum	Maximum
1 inch	99	100
½ inch	90	100
¼ inch	40	90
#200	2	10

Tests should be sufficiently recent to represent the actual material that is anticipated to be delivered to the site. If processes or sources used by the supplier have changed significantly since the most recent testing, new tests should be requested.

Note: the gradation of compost used in bioinfiltration media is believed to play an important role in the saturated hydraulic conductivity of the media. To achieve a higher saturated hydraulic conductivity, it may be necessary to utilize compost at the coarser end of this range ("minimum" column). The percent passing the #200 sieve (fines) is believed to be the most important factor in hydraulic conductivity.

In addition, a coarser compost mix provides more heterogeneity of the bioinfiltration media, which is believed to be advantageous for more rapid development of soil structure needed to support health biological processes. This may be an advantage for plant establishment with lower nutrient and water input.

5) The bioinfiltration area should be covered with 2 to 4 inches (average 3 inches) of mulch at the start and an additional placement of 1 to 2 inches of mulch should be added annually. *The intention is that to help sustain the nutrient levels, suppress weeds, retain moisture, and maintain infiltration capacity*.

Planting/Storage Media Design for Nutrient Sensitive Receiving Waters

1) Where the BMP discharges to receiving waters with nutrient impairments or nutrient TMDLs, the planting media placed in the cell should be designed with the specific goal of minimizing the potential for initial and long term leaching of nutrients from the media.

2) In general, the potential for leaching of nutrients can be minimized by:

    a. Utilizing stable, aged compost (as required of media mixes under all conditions).

    b. Utilizing other sources of organic matter, as appropriate, that are safe, non-toxic, and have lower potential for nutrient leaching than compost.

    c. Reducing the content of compost or other organic material in the media mix to the minimum amount necessary to support vigorous plant growth and healthy biological processes.

3) A landscape architect should be consulted to assist in the design of planting/storage media to balance the interests of plant establishment, water retention capacity (irrigation demand), and the potential for nutrient leaching. The following practices should be considered in developing the media mix design:

    a. The actual nutrient content and organic content of the selected compost source should be considered when specifying the proportions of compost and sand. The compost specification allows a range of organic content over approximately a factor of 2 and nutrient content may vary more widely. Therefore determining the actual organic content and nutrient content of the compost expected to be supplied is important in determining the proportion to be used for amendment.

    b. A commitment to periodic soil testing for nutrient content and a commitment to adaptive management of nutrient levels can help reduce the amount of organic amendment that must be provided initially. Generally, nutrients can be added planting areas through the addition of organic mulch, but cannot be removed.

    c. Plant palettes and the associated planting mix should be designed with native plants where possible. Native plants generally have a broader tolerance for nutrient content, and can be longer lived in leaner/lower nutrient soils. An additional benefit of lower nutrient levels is that native plants will generally have less competition from weeds.

d. Nutrients are better retained in soils with higher cation exchange capacity (CEC). CEC can be increased through selection of organic material with naturally high CEC, such as peat, and/or selection of inorganic material with high CEC such as some sands or engineered minerals (e.g., low P-index sands, zeolites, rhyolites, etc). Including higher CEC materials would tend to reduce the net leaching of nutrients.

e. Soil structure can be more important than nutrient content in plant survival and biologic health of the system. If a good soil structure can be created with very low amounts of compost, plants survivability should still be provided. Soil structure is loosely defined as the ability of the soil to conduct and store water and nutrients as well as the degree of aeration of the soil. While soil structure generally develops with time, planting/storage media can be designed to promote earlier development of soil structure. Soil structure is enhanced by the use of amendments with high hummus content (as found in well-aged organic material). In addition, soil structure can be enhanced through the use of compost/organic material with a distribution of particle sizes (i.e., a more heterogeneous mix). Finally, inorganic amendments such as polymer beads may be useful for promoting aeration and moisture retention associated with a good soil structure. An example of engineered soil to promote soil structure can be found here:

   http://www.hort.cornell.edu/uhi/outreach/pdfs/custructuralsoilwebpdf.pdf

f. Younger plants are generally more tolerant of lower nutrient levels and tend to help develop soil structure as they grow. Starting plants from smaller transplants can help reduce the need for organic amendments and improve soil structure. The project should be able to accept a plant mortality rate that is somewhat higher than starting from larger plants and providing high organic content.

g. With these considerations, it is anticipated that less than 10 percent compost amendment could be used, while still balancing plant survivability and water retention.

Plants

1) Plant materials should be tolerant of summer drought, ponding fluctuations, and saturated soil conditions for 48 to 96 hours.

2) It is recommended that a minimum of three types of tree, shrubs, and/or herbaceous groundcover species be incorporated to protect against facility failure due to disease and insect infestations of a single species.

3) Native plant species and/or hardy cultivars that are not invasive and do not require chemical inputs should be used to the maximum extent practicable.

Operations and Maintenance

Bioinfiltration areas require annual plant, soil, and mulch layer maintenance to ensure optimum infiltration, storage, and pollutant removal capabilities. In general, bioinfiltration maintenance requirements are typical landscape care procedures and include:

1) Watering: Plants should be drought-tolerant. Watering may be required during prolonged dry periods after plants are established.
2) Erosion control: Inspect flow entrances, ponding area, and surface overflow areas periodically, and replace soil, plant material, and/or mulch layer in areas if erosion has occurred (see Appendix I for a bioinfiltration inspection and maintenance checklist). Properly designed facilities with appropriate flow velocities should not have erosion problems, except perhaps in extreme events. If erosion problems occur, the following should be reassessed: (1) flow velocities and gradients within the cell, and (2) flow dissipation and erosion protection strategies in the pretreatment area and flow entrance. If sediment is deposited in the bioinfiltration area, immediately determine the source within the contributing area, stabilize, and remove excess surface deposits.
3) Plant material: Depending on aesthetic requirements, occasional pruning and removing of dead plant material may be necessary. Replace all dead plants and if specific plants have a high mortality rate, assess the cause and, if necessary, replace with more appropriate species. Periodic weeding is necessary until plants are established. The weeding schedule should become less frequent if the appropriate plant species and planting density have been used and, as a result, undesirable plants excluded.
4) Nutrients and pesticides: The soil mix and plants should be selected for optimum fertility, plant establishment, and growth. Nutrient and pesticide inputs should not be required and may degrade the pollutant processing capability of the bioinfiltration area, as well as contribute pollutant loads to receiving waters. By design, bioinfiltration facilities are located in areas where phosphorous and nitrogen levels are often elevated and these should not be limiting nutrients. If in question, have soil analyzed for fertility.
5) Mulch: Replace mulch annually in bioinfiltration facilities where heavy metal deposition is likely (e.g., contributing areas that include industrial and auto dealer/repair parking lots and roads). In residential lots or other areas where metal deposition is not a concern, replace or add mulch as needed to maintain a 2 to 3 inch depth at least once every two years.
6) Soil: Soil mixes for bioinfiltration facilities are designed to maintain long-term fertility and pollutant processing capability. Estimates from metal attenuation research suggest that metal accumulation should not present an environmental

concern for at least 20 years in bioinfiltration systems. Replacing mulch in bioinfiltration facilities where heavy metal deposition is likely provides an additional level of protection for prolonged performance. If in question, have soil analyzed for fertility and pollutant levels.

RWH-1: Rainwater Harvesting

Rainwater harvesting BMPs capture and store stormwater runoff for later use. These BMPs are engineered to store a specified volume of water with no surface discharge until this volume is exceeded. Storage facilities that can be used to harvest rainwater include cisterns (above ground tanks), open storage reservoirs (e.g., ponds and lakes), and underground storage devices (tanks, vaults, pipes, arch spans, and proprietary storage systems). Uses of captured water may potentially include irrigation demand, indoor non-potable demand, industrial process water demand, or other demands. Rainwater harvesting systems typically include several components: (1) methods to divert runoff to the storage device, (2) an overflow for when the storage device is full, and (3) a distribution system to get the water to where it is intended to be used. Harvesting systems typically include pretreatment to remove large sediment and vegetative debris. Systems used for internal uses may require an additional level of treatment prior to use.

Cistern

Photo Credit: MetaEfficient

Application

- Any type of land use, provided adequate water demand

Preventative Maintenance

- Debris and sediment removal
- After-rain inspections

Limitations

Rainwater harvesting may be used to meet all of the 5% EIA requirement if reliable demand is available. Rainwater harvesting is not required to be used if the available demands do not meet the volume required for 80% capture using a 72 hour drawdown time.

Design Criteria

Specific considerations for cistern rainwater harvesting systems include:

- Cisterns should include screens on gutters and downspouts to remove vegetative debris and sediment from the runoff prior to entering the cistern.
- Above-ground cisterns should be secured in place.
- Above-ground cisterns should not be located on uneven or sloped surfaces; if installed on a sloped surface, the base where the cistern will be installed should be leveled and designed for the weight of the filled cistern prior to installation.
- Child-resistant covers and mosquito screens should be placed on all water entry holes.
- A first flush diverter may be installed so that initial runoff bypasses the cistern. Where a first flush diverter is used, the diverted flows must be directed to a pervious area so that no runoff is produced or another form of treatment must be provided for this flow.
- Above-ground cisterns should be installed in a location with easy access for maintenance or replacement.

Specific considerations for underground detention include:

- Access entry covers (36” diameter minimum) should be locking and within 50 feet of all areas of the detention tank.
- In cases where the detention facility provides sediment containment, the facility should be laid flat and there should be at least ½ foot of dead storage within the tank or vault.
- Outlet structures should be designed using the 100-year storm as overflow and should be easily accessible for maintenance activities.
- For detention facilities beneath roads and parking areas, structural requirements should meet H20 load requirements.
- In cases where groundwater may cause flotation, these forces should be counteracted with backfill, anchors, or other measures.

- Underground detention facilities should be installed on consolidated and stable native soil; if the facility is constructed in fill slopes, a geotechnical analysis should be performed to ensure stability.

General considerations include:

- In cases where there is non-potable indoor demand, proper pretreatment measures should be installed such as pre-filtration, cartridge filtration, and/or disinfection (which can also be provided between the cistern and point of use).
- Plumbing systems should be installed in accordance with the current California Building and Plumbing Codes (CBC – part of California Code of Regulations, Title 24).
- Underground detention facilities can be incorporated into a treatment train to provide initial or supplemental storage to other detention storage facilities and/or infiltration BMPs.
- Treatment of the captured rainwater (i.e. disinfection) may be required depending on the end use of the water.

Rainwater harvesting uses include:

- Harvested rainwater can be used for irrigation and other non-potable uses (if local, State, and Federal ordinances allow). The use of captured stormwater allows a reduced demand on the potable water supply. Cross-contamination should be prevented when make-up water is required for rainwater use demand by providing a backflow prevention system on the potable water supply line and/or an air gap.
- Irrigation Use
    - Subsurface (or drip) irrigation should not require disinfection pretreatment prior to use; other irrigation types, such as spray irrigation, may require additional pre-treatment prior to use
    - Selecting native and/or drought tolerant plants for landscaped area will reduce irrigation demand; however, they are still recommended for use.
- Domestic Use
    - Domestic uses may include toilet flushing and clothes washing (if local, State, and Federal ordinances allow).
    - Pretreatment requirements per local, State, or Federal codes and ordinances may apply.
- Other Non-Potable Uses

- Other potential non-potable uses may include vehicle/equipment washing, evaporative cooling, industrial processes, and dilution water for recycled water systems.

Sizing Criteria

The effectiveness of rainwater harvesting (RWH) systems is a function of tributary area, storage volume, demand patterns and magnitudes, and operational regime. If either of the latter two factors are too complex, simple design criteria metrics are not possible. The rainwater harvesting design criteria provided in this Fact Sheet are intended for the evaluation of systems that have relatively simple demand regimes and passive operation. If the answer to any of the following complexity screening questions is yes, a site-specific evaluation of rainwater harvesting effectiveness should be completed using a continuous simulation model with a long-term precipitation record.

Complexity Screening Questions:

- Does the proposed system have seasonally-varying demand other than irrigation?
- Will the system be operated by advanced control systems or otherwise actively controlled?
- Does the operational regime call for the system be shut down at any time during the rainy season?

Effectiveness of a harvesting system for retaining the SQDV depends on the cistern's effective storage capacity (i.e., the volume available for storage at the beginning of each event). Therefore, the required storage volume varies based on precipitation and demand. Using the following sizing charts, cisterns should be sized to achieve 80 percent capture efficiency. These nomographs are based on continuous simulation performed in EPA SWMM using precipitation and ET records representative of lowland regions (Oxnard Airport Precipitation Gauge, El Rio Spreading Grounds ET station) and mountainous regions (Ojai-Stewart Canyon Precipitation Gauge, Matilja ET Station) of the County.

Instructions for determining required cistern volume and demand are provided below:

Step 1: Determine Required Rainwater Harvesting Design Volume (RWHDV)

Note that a rainwater harvesting system sized for 80% capture runoff (as determined by continuous modeling), which can draw down in 72 hours is required to meet the 5% EIA standard. If the demand required to draw a tank sized for these parameters is not available, rainwater harvesting is not mandated for use. Partial capture of runoff is allowable if rainwater harvesting is desired for use. Sizing instructions for partial capture are included in Step 3.

1) Determine the design storm required for 80% capture with a 72 hour drawdown time by selecting the project region (lowland or mountainous), then determining where the 72 hour drawdown curve intersects the 80% capture line. Pivot down from this intersection to the x axis to read the design storm, d_{design}.

2) Determine the required rainwater harvesting system volume using the following equation:

$$RWHDV = C^*(d_{design}/12)^*A_{retain} \qquad \text{(Equation 6-19)}$$

Where:

RWHDV	=	rainwater harvesting design volume (acre-ft)
C	=	runoff coefficient, calculated using Appendix E and the site imperviousness
d_{design}	=	design storm required for 80% capture with a 72 hour drawdown time, estimated as described in 1) (inches)
A_{retain}	=	the drainage area from which runoff must be retained (acres)

Step 2: Determine the Required Daily Demand to Achieve 80% Capture

1) The required daily demand to achieve 80% capture of runoff can be calculated as follows:

$$\text{Demand} = [RWHDV/(72/24)] * (325{,}851) \qquad \text{(Equation 6-20)}$$

Where:

Demand	=	required project daily demand to draw down rainwater harvesting system sized for 80% capture in 72 hours (gallons)
RWHDV	=	rainwater harvesting design volume (acre-ft), from Step 1 above

If the project daily demand is less than the Demand calculated, the project is not required to utilize rainwater harvesting. If rainwater harvesting is desired for use for partial retention, if a longer drawdown time is desired, or if a predetermined daily demand is to be used, refer to Steps 3 and 4 below.

Step 3: Determine RWHDV for Partial Retention or a Longer Drawdown Time

1) Calculate RWHDV for selected combination of % capture and drawdown time using nomographs and the following equation:

$$RWHDV = C*(d_{design}/12)*A_{retain} \qquad \text{(Equation 6-21)}$$

Where:

RWHDV	=	rainwater harvesting design volume (acre-ft)
C	=	runoff coefficient, calculated using Appendix E and the site imperviousness
d_{design}	=	design storm required for selected % capture and drawdown time (inches)
A_{retain}	=	the drainage area from which runoff must be retained (acres)

2) Determine the required daily demand for the selected capture efficiency and/or drawdown time:

$$Demand = [RWHDV/(t_{drawdown}/24)] * (325{,}851) \qquad \text{(Equation 6-22)}$$

Where:

Demand	=	required project daily demand to draw down rainwater harvesting system sized for 80% capture in 72 hours (gallons)
RWHDV	=	rainwater harvesting design volume (acre-ft), from 1) above
$t_{drawdown}$	=	selected drawdown time (hours)

Step 4: Determine RWHDV for a Predetermined Daily Demand

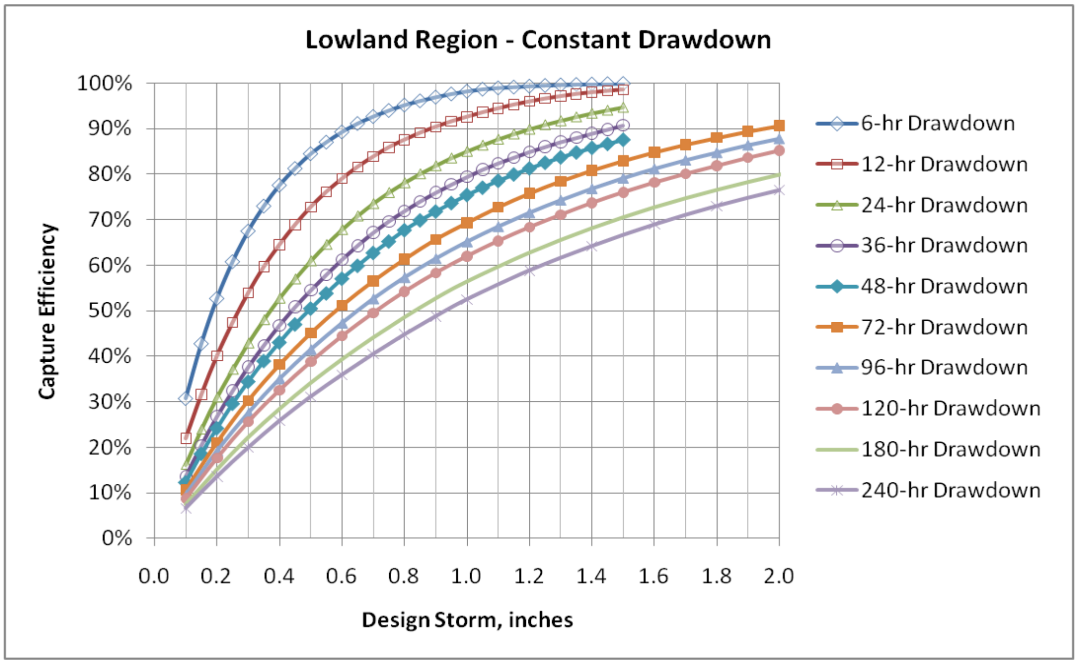
1) Determine the daily demand requirement in acre-feet (1 acre-foot = 325,851 gallons).

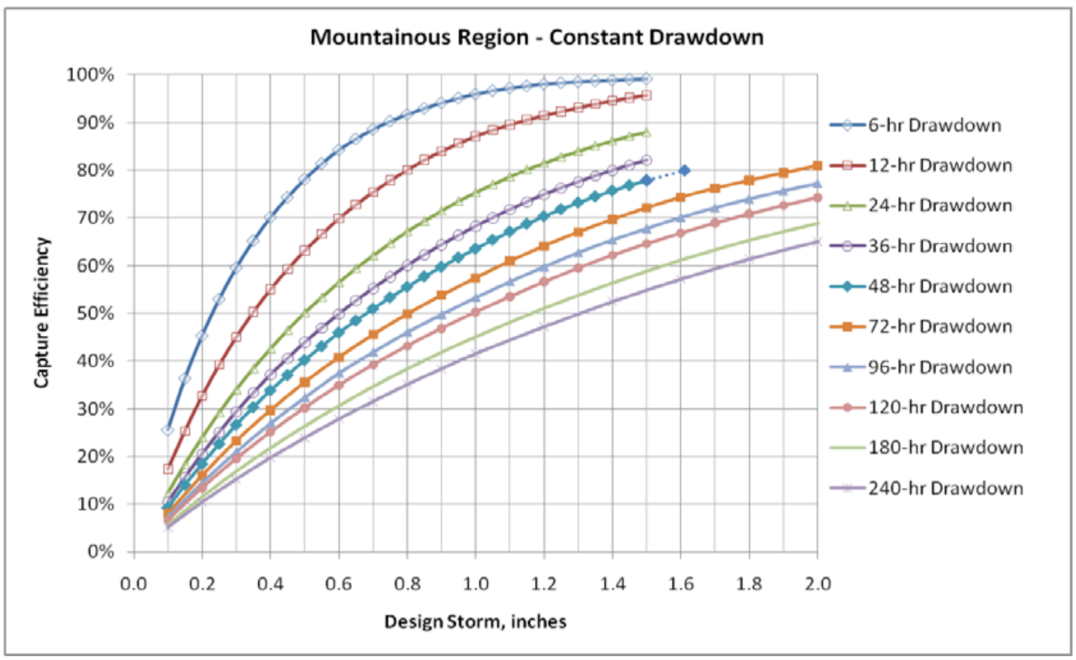
2) Calculate the required RWHDV for the desired drawdown time using the following equation:

$$RWHDV = Demand *(t_{drawdown}/24) \qquad \text{(Equation 6-23)}$$

Where:

Demand	=	required project daily demand (acre-feet)
RWHDV	=	rainwater harvesting design volume (acre-ft)
$t_{drawdown}$	=	selected drawdown time (hours)


Lowland Region - Constant Drawdown
Capture Efficiency
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Design Storm, inches
6-hr Drawdown
12-hr Drawdown
24-hr Drawdown
36-hr Drawdown
48-hr Drawdown
72-hr Drawdown
96-hr Drawdown
120-hr Drawdown
180-hr Drawdown
240-hr Drawdown


Mountainous Region - Constant Drawdown
Capture Efficiency
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
Design Storm, inches
6-hr Drawdown
12-hr Drawdown
24-hr Drawdown
36-hr Drawdown
48-hr Drawdown
72-hr Drawdown
96-hr Drawdown
120-hr Drawdown
180-hr Drawdown
240-hr Drawdown

Operations and Maintenance

1) Inspect storage facilities, associated pipes, and valve connections for leaks.
2) Clean gutters and filters of debris that has accumulated and is obstructing flow into the storage facility.
3) Clean and remove accumulated sediment annually.
4) Check cisterns for stability and anchor if necessary.
5) If the storage device is underground, ensure that a manhole is accessible, operational, and secure.

ET-1: Green Roof

Green roofs (also known as eco-roofs and vegetated roof covers) are roofing systems that layer a soil/vegetative cover over a waterproofing membrane. Green roofs rely on highly porous media and moisture retention layers to store intercepted precipitation and to support vegetation that can reduce the volume of stormwater runoff via evapotranspiration. There are two types of green roofing systems: extensive, which is a light-weight system; and intensive, which is a heavier system that allows for larger plants but requires additional structural support.

Green Roof Examples

Photo Credits:

1. Milwaukee Department of Environmental Sustainability;

2. Geosyntec Consultants

Application

- Building roofs
- Outdoor eating area roofs
- Parking structure or turnaround roofs

Preventative Maintenance

- Weeding and pruning
- Leaf and debris removal
- Regular membrane inspection
- Drain cleanout

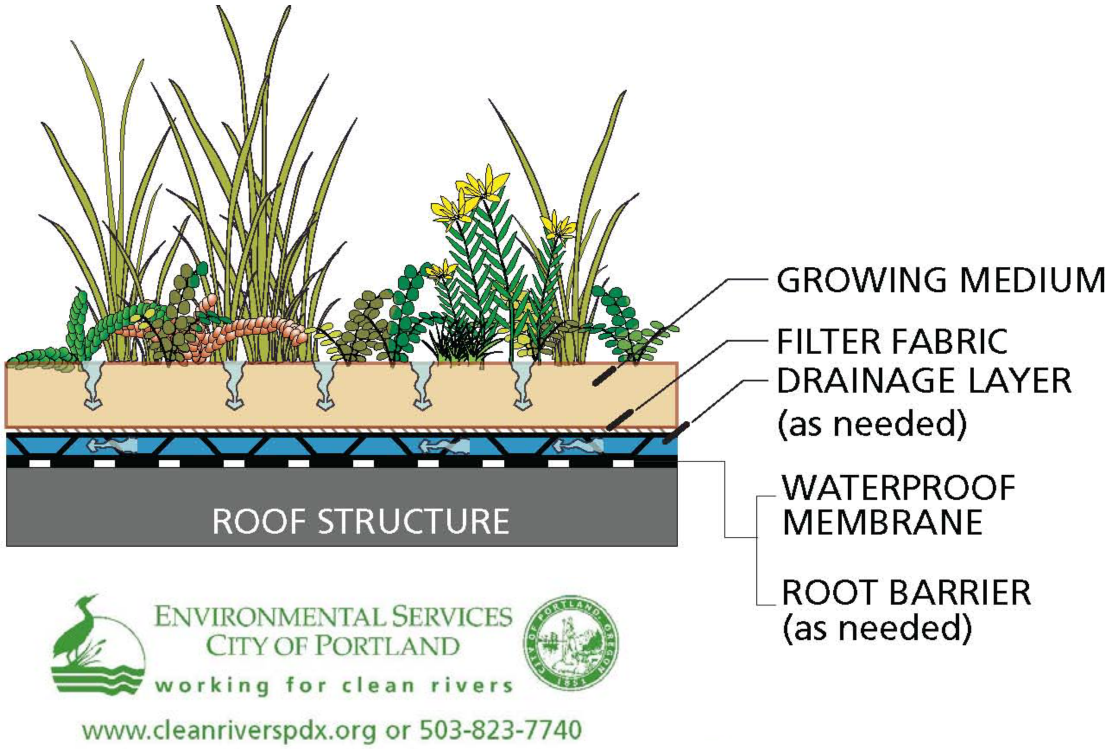


Exhibit A: Green Roof Schematic Courtesy of Portland, OR Environmental Services Department

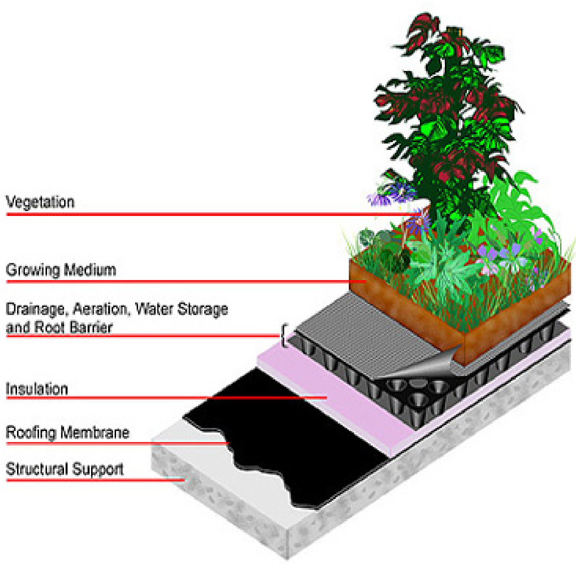


Exhibit B: Green Roof Schematic Courtesy of American Wick

Geosyntec
consultants

Figure 6-9: Green Roofs

Limitations

The following describes additional site suitability recommendations and limitations for green roofs.

- Typically not used for steep roofs (>25%); and
- Structural roof support must be sufficient to support additional roof weight.

Design Criteria

Green roofs should be designed according to the requirements listed in Table 6-17 and outlined in the section below.

Table 6-17: Green Roof Design Criteria

Design Parameter	Unit	Design Criteria
Soil depth range	inch	2 – 6
Saturated soil weight	lbs. / sq. ft.	10 – 25
Maximum roof slope	%	25
Minimum roof slope	--	Flat
Vegetation type	--	Varies (see vegetation section below)
Vegetation height	--	Varies (see vegetation section below)

Sizing

Green roofs may provide quantifiable reduction in volume. However, they are not explicitly sized to meet the water quality treatment requirements. Rather, the volume reduction is accounted for implicitly in sizing calculations for the treatment BMPs for the remainder of the site by assuming that the roof area is pervious rather than impervious when calculating a runoff coefficient for the site.

Green Roof Components

Structural Support

The first requirement that must be met before installing a green roof is the structural support of the roof. The roof must be able to support the additional weight of the soil, water, and vegetation. A licensed structural engineer should be consulted to determine the proposed structural support during the design phase.

Waterproof Roofing Membrane

Waterproof roofing membrane is an integral part of a green roofing system. The waterproof membrane prevents the roof runoff from penetrating and damaging the roofing material. There are many materials available for this purpose and come in various forms (i.e., rolls, sheets, liquid) and exhibit different characteristics (e.g., flexibility, strength, etc.). Depending on the type of membrane chosen a root barrier may be required to prevent roots from compromising the integrity of the membrane.

Drainage Layer

Depending on the design of the roof, a drainage layer may be required to convey the excess runoff from of the roof. If a drainage layer is needed, there are numerous options including a gravel layer (which may require additional structural support), and many styles and types of plastic drainage layers.

Soil Considerations

The soil layer is an important factor in the construction and operation of green roofs. The soil layer must have excellent drainage, not be too heavy when saturated, and be adequately fertile as a growing medium for plants. Many companies sell their own proprietary soil mixes. However, a simple mix of ¼ topsoil, ¼ compost, and the remainder pumice perlite may be used for many applications. Other soil amendments may be substituted for the compost and the pumice perlite. The soil mix used should not contain any clay.

Vegetation

Green roofs must be vegetated in order to provide adequate treatment of runoff via filtration and evapotranspiration. Vegetation, when chosen and maintained appropriately, also improves the aesthetics of a site. Green roofs should be vegetated with a mix of erosion-resistant plant species that effectively bind the soil and can withstand the extreme environment of rooftops. A diverse selection of low growing plants that thrive under the specific site, climatic, and watering conditions should be identified. A mixture of drought-tolerant, self-sustaining (perennial or self-sowing without need for fertilizers, herbicides, and or pesticides) is most effective in the Ventura County region. Plants selected should also be low maintenance and able to withstand heat, cold, and high winds. Native or adapted sedum/succulent plants are preferred because they generally require less fertilizer, limited maintenance, and are more drought resistant than exotic plants. When appropriate, green roofs may be planted with larger plants. However, this depends on structural support and soil depth.

The following provides additional vegetation guidance for green roofs.

1) For extensive roofs, trees or shrubs may be used as long as the increased soil depth required may be supported.

2) Irrigation is required if the seed is planted in spring or summer. The use of a permanent smart (self-regulating) irrigation system or other watering system, may help provide maximal water quality performance. Drought-tolerant plants should be specified to minimize irrigation requirements. For projects seeking "High Performance Building" recognition, ASHRAE Standard 189.1 states that potable water cannot be used for irrigating green roofs after they are established.

3) Locate the green roof vegetation in an area without excessive shade to avoid poor vegetative growth. For moderately shaded areas, shade tolerant plants should be used.

4) A relevant plant list should be provided by a landscape professional and used as a guide to support project-specific planting recommendations, including recommendations on appropriate plants, fertilizer, mulching applications, and irrigation requirements (if any) to ensure healthy vegetation growth.

Drain

1) There must be a drain pipe (gutter) to convey runoff (both overflow and underdrain flow, if appropriate) safely from the roof to another basic or stormwater runoff BMP, a pervious area, or the stormwater conveyance system.

Construction Considerations

1) Building structure must be adequate to hold the additional weight of the soil, retained water, and plants.

2) Plants should be selected carefully to minimize maintenance and function properly.

Operations and Maintenance

1) During the establishment period, green roofs may need irrigation and occasional light fertilization until the plants have fully established themselves. Once healthy and fully established, properly selected climate-appropriate plants will no longer need irrigation except during extreme drought.

2) Weeding during the establishment period may be required to ensure proper establishment of the desired vegetation. Once established and assuming proper selection of vegetation, the vegetation should not require any preventative maintenance.

3) The roofing membrane should be inspected routinely, as it is a crucial element of the green roof. In addition, preventative inspection of the drainage paths is required to ensure that there are no clogs in the system. If a green roof is not properly draining, the moisture in the system may cause the roof to leak and/or the plants to drown or rot. Leaks in the roof may occur not only due to improper drainage, but also if the incorrect combination of waterproofing barrier, root barrier, and drainage systems

are selected. Leak inspections in the roofing system are advised, especially in locations prone to leaks, such as at all joints.

4) Inspect green roofs for erosion or damage to vegetation after every storm greater than 0.75 inches and at the end of the wet season to schedule summer maintenance and in the fall to ensure readiness for winter. Additional inspection after periods of heavy runoff is recommended. Green roofs should be checked for debris, litter, and signs of clogging.

5) Replanting and/or reseeding of vegetation may be required for reestablishment.

6) Vegetation should be healthy and dense enough to provide filtering while protecting underlying soils from erosion.

7) Fallen leaves and debris from deciduous plant foliage should be removed.

8) Invasive vegetation, such as Alligatorweed (*Alternanthera philoxeroides*), Halogeton (*Halogeton glomeratus*), Spotted Knapweed (*Centaurea maculosa*), Giant Reed (*Arundo donax*), Castor Bean (*Ricinus communis*), Perennial Pepperweed (*Lepidium latifolium*), and Yellow Starthistle (*Centaurea solstitalis*) should be removed and replaced with non-invasive species. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website or the California Invasive Plant Council website at www.cal-ipc.org.

9) Dead vegetation should be removed if greater than 10% of the area coverage. Vegetation should be replaced and established before the wet season to maintain cover density and control erosion where soils are exposed.

ET-2: Hydrologic Source Control BMPs

Hydrologic source control (HSC) BMPs are simple BMPs that are highly integrated with the site design to reduce runoff volume. The practices described in this fact sheet include impervious area dispersion, street trees, and rain barrels.

Application

- Building roofs
- Sidewalks and patios
- Landscaping hardscapes

Preventative Maintenance

- Weeding and pruning
- Leaf and debris removal

Hydrologic Source Control Examples

Photo Credits:

1. http://www.auburn.edu/projects/sustainability/website/newsletter/0910.php;

2. Geosyntec Consultants;

3. toronto.ca/environment/water.htm

Accounting for Hydrologic Source Controls in Hydrologic Calculations

The effects of HSC BMPs are accounted for in hydrologic calculations as an adjustment to the storm depth used in the SQDV calculations described in Section 2. Runoff volume calculations are performed exactly as described in Section 2, with the exception that the storm depth used in the calculation is adjusted prior to the calculation. Adjustments are based on the type and magnitude of HSC BMPs employed for the drainage area per guidance outlined in this Fact Sheet.

EXAMPLE 6.1: ACCOUNTING FOR HSCS IN HYDROLOGIC CALCULATIONS

Given:

- A drainage area consists of a 1 acre building roof surrounded by 0.25 acres of landscaping (80 percent composite imperviousness);
- The drainage from the roof is spread uniformly over the entire pervious area via splash pads and level spreaders;
- Soils are moderately well drained and have a shallow slope;
- For the purpose of this example, assume the hydrologic source control adjustment for this configuration of disconnected downspouts is 0.3 inches. For an actual project, hydrologic source control adjustment would be calculated based on instructions in this section; and
- The unadjusted design storm depth at the project site is 0.75 inches.

Result:

1) The designer uses 0.75 inches – 0.3 inches = 0.45 inches in the calculation of SQDV.

Impervious Area Dispersion

Impervious area dispersion refers to the practice of routing runoff from impervious areas, such as rooftops, walkways, and patios, onto the surface of adjacent pervious areas. Runoff is dispersed uniformly via splash block or dispersion trench and soaks into the ground as it moves slowly across the surface of the pervious area. Minor ponding may occur, but it is not the intent of this practice to actively promote localized on-lot infiltration, which should be designed as an infiltration BMP (see INF-1 through INF-6 above).

Design Considerations

1) Not likely to result in net increased infiltration over existing condition for previously pervious sites, but has potential to result in some geotechnical hazards associated with infiltration.

2) Significant pervious area should be available, at a ratio of at least 1 part pervious area capable of receiving flow to 5 parts impervious.

3) Pervious area receiving flow should have a slope ≤ 2 percent and path lengths of ≥ 10 feet per 1000 sf of impervious area.

4) Overflow from the pervious area up to the SQDV should be directed to a Retention BMP, Biofiltration BMP, or Treatment Control Measure. Larger flows should be directed to the storm drain system.

5) Soils in the pervious area should be preserved in their natural condition or improved with soil amendments (see Soil Amendments below).

6) Impervious area disconnection is an HSC that may be used as the first element in any treatment train.

7) The use of impervious area disconnection reduces the sizing requirement for downstream Retention BMPs, Biofiltration BMPs, and/or Treatment Control Measures.

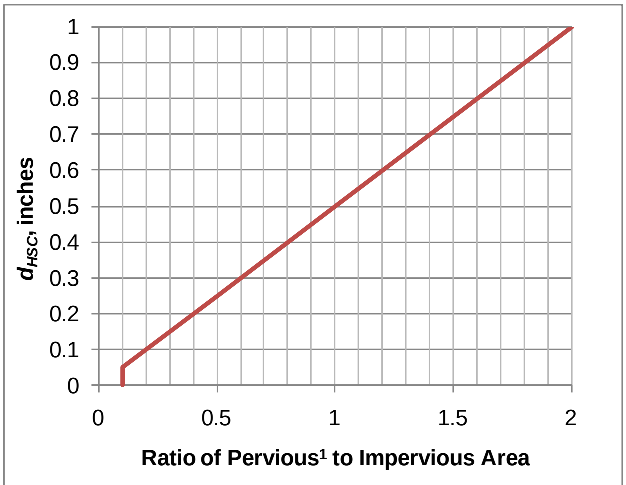
Calculating HSC Retention Volume

1) The retention volume provided by downspout dispersion is a function of the ratio of impervious to pervious area.

2) Determine flow patterns in pervious area and estimate footprint of pervious area receiving dispersed flow. Calculate the ratio of pervious to impervious area.

3) Check soil conditions using the checklist below; amend if necessary.

4) Look up the storm retention depth (d_{HSC}), from the chart to the right.



[1] Pervious area used in calculation should only include the pervious area receiving flow, not pervious area receiving only direct rainfall or upslope pervious drainage.

5) The max d_{HSC} is equal to the design storm depth for the project site.

Soil Condition Checklist

1) Soil should have a maximum slope of 2 percent.

2) Landscaping should be well-established.

3) Amended soils should consist of: 60 to 70% sand, 15 to 25% compost, 10 to 20% clean topsoil. The organic content of the soil mixture should be 8 to 12%; the pH range should be 5.5 to 7.5.

Additional References

- SMC LID Manual (pp 131): http://www.lowimpactdevelopment.org/guest75/pub/All_Projects/SoCal_LID_Manual/SoCalLID_Manual_FINAL_040910.pdf

- City of Portland Bureau of Environmental Services. 2010. How to manage stormwater – Disconnect Downspouts: http://www.portlandonline.com/bes/index.cfm?c=43081&a=177702

- Seattle Public Utility: http://www.cityofseattle.org/util/stellent/groups/public/@spu/@usm/documents/webcontent/spu01_006395.pdf

- Thurston County, Washington State (pp 10): http://www.co.thurston.wa.us/wwm/Engineering_Standards/Drainage_Manual/PDFs/DG-5%20Roof%20Runoff%20Control.pdf

Amended Soils

A soil amendment is any material added to the upper layer of soil especially in the vicinity of the root zone soil to improve its physical properties, such as the water retention, permeability, water infiltration, drainage, aeration and structure. The goal is to provide a better environment for roots. To do its work, an amendment should be thoroughly mixed into the soil. If it is merely buried, its effectiveness is reduced and it will interfere with water and air movement and root growth.

Amending a soil is different from mulching, although many mulches also are used as amendments. A mulch is left on the soil surface. Its purpose is to reduce evaporation and runoff, inhibit weed growth, and create an attractive appearance. Mulches also moderate soil temperature, helping to warm soils in the spring and cool them in the summer. Mulches may be incorporated into the soil as amendments after they have decomposed to the point that they no longer serve their purpose.

Organic amendments, such as compost, increase soil organic matter content and offer many benefits. Organic matter improves soil aeration, water infiltration, and both water- and nutrient-holding capacity. Many organic amendments contain plant nutrients and act as organic fertilizers. Organic matter also is an important energy source for bacteria, fungi and earthworms that live in the soil.

Design Considerations

1) Landscaped and other developed pervious areas can be amended to improve evapotranspiration and soil moisture storage capacity.

2) Landscape and other developed pervious areas can be amended to increase infiltration rates in cases where the limiting infiltration horizon exists near the surface of the soil column.

3) Soil amendments are common components of several Retention BMPs, Biofiltration BMPs, and Treatment Control Measures, including infiltration basins, bioretention, vegetated swales, filter strips, planter boxes, green roofs, dry extended detention basins, wet retention basins, and constructed treatment wetlands.

4) Compost, soil conditioners, and fertilizers should be rototilled into the native soil to a minimum depth of 6 inches; 12 inches preferred.

5) All soil amendments shall be free of sticks, glass, plastic, metal, debris larger than 1 inch, and other deleterious material.

6) Compost shall meet criteria listed in the guidelines for planting and storage media.

Calculating HSC Retention Volume

No retention credit is given for amended soils alone. Amended soils should be used to increase the retention volume of Retention BMPs, Biofiltration BMPs, and Treatment Control Measures.

Additional References

- San Diego County LID Handbook Appendix 4 (Factsheet 30): http://www.sdcounty.ca.gov/dplu/docs/LID-Appendices.pdf

- Colorado State University Extension website: http://www.ext.colostate.edu/pubs/garden/07235.html

Street Trees

By intercepting rainfall, trees can provide several aesthetic and stormwater benefits including peak flow control, increased infiltration and evapotranspiration, and runoff temperature reduction. The volume of precipitation intercepted by the canopy reduces the treatment volume required for downstream treatment BMPs. Shading reduces the heat island effect as well as the temperature of adjacent impervious surfaces over which stormwater flows, and thus reduces the heat transferred to the downstream waterbody. Tree roots also strengthen the soil structure and provide infiltrative pathways, simultaneously reducing erosion potential and enhancing infiltration.

Design Considerations

1) Street trees can be incorporated along sidewalks, streets, parking lots, or driveways.

2) Street trees can be used in combination with bioretention systems along medians or in traffic calming bays.

3) There should be sufficient space available to accommodate both the tree canopy and the root system.

4) The mature tree canopy, height, and root system should not interfere with subsurface utilities, overhead powerlines, buildings and foundations, or other existing or planned structures.

5) Depending on space constraints, a 20 to 30 foot canopy (at maturity) is recommended for stormwater mitigation.

6) Native, drought-tolerant species should be selected in order to minimize irrigation requirements and improve the long-term viability of the tree.

7) Trees should not impede pedestrian or vehicle sight lines.

8) Planting locations should receive adequate sunlight and wind protection. Other environmental factors should be considered prior to planting.

9) Soils should be preserved in their natural condition (if appropriate for planting) or restored via soil amendments. If necessary, a landscape architect should be consulted.

Calculating HSC Retention Volume

1) The retention volume provided by streets trees via canopy interception is dependent on the tree species, time of the year, and maturity.

2) To compute the retention credit, the expected impervious area covered by the full tree canopy after 4 years of growth should be computed (IA_{HSC}). The maximum retention depth credit for canopy interception (d_{HSC}) is 0.05 inches.

Additional References

- California Stormwater BMP Handbook: http://www.cabmphandbooks.com/Documents/Development/Section_3.pdf
- City of Los Angeles, Street Tree Division - Street Tree Selection Guide: http://bss.lacity.org/UrbanForestryDivision/StreetTreeSelectionGuide.htm
- Portland Stormwater Management Manual: http://www.portlandonline.com/bes/index.cfm?c=35122&a=55791
- San Diego County LID Handbook Fact Sheets: http://www.sdcounty.ca.gov/dplu/docs/LID-Appendices.pdf

Residential Rain Barrels

Rain barrels are above ground storage vessels that capture runoff from roof downspouts during rain events and detain that runoff for later use for irrigating landscaped areas.

Design Considerations

1) If detained water will be used for irrigation, sufficient vegetated areas and other impervious surfaces should be present in the drainage area.

2) Storage capacity and sufficient area for overflow dispersion should be accounted for.

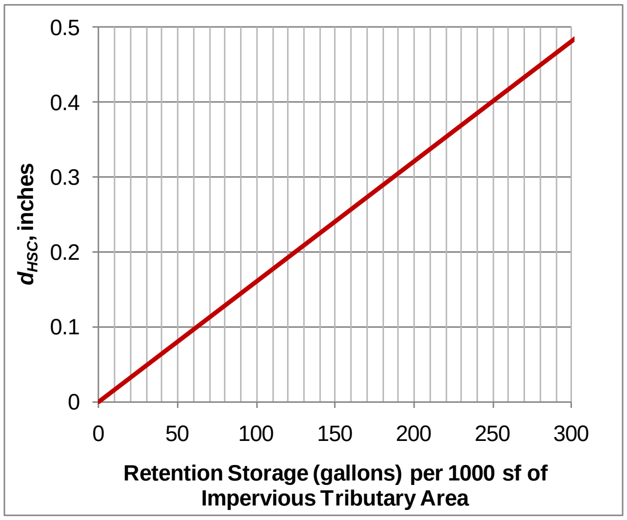
3) Screens on gutters and downspouts to remove sediment and particles as the water enters the barrel or cistern should be provided.

4) Removable child-resistant covers and mosquito screening should be provided to prevent unwanted access.

5) Above-ground barrels should be secured in place.

6) Above-ground barrels should not be located on uneven or sloped surfaces. If installed on a sloped surface, the base where the rain barrel will be installed should be leveled prior to installation.

7) Overflow dispersion should occur greater than 5 feet from building foundations.



8) Dispersion should not cause geotechnical hazards related to slope stability.

9) Effective energy dissipation and uniform flow spreading methods should be employed to prevent erosion and facilitate dispersion.

10) Placement should allow easy access for regular maintenance.

Calculating HSC Retention Volume

1) The retention volume provided by rain barrels that are not actively managed can be computed as 50% of the total storage volume (e.g., 22.5 gallons for each 55 gallon barrel).

2) If the rain barrel is actively managed, then it should be treated as a cistern (see RWH-1).

3) Estimate the average retention volume per 1000 square feet impervious tributary area provided by rain barrels.

4) Look up the storm retention depth (d_{HSC}), from the chart to the right.

5) The max d_{HSC} is equal to the design storm depth for the project site.

Additional References

- Santa Barbara BMP Guidance Manual, Chapter 6: http://www.santabarbaraca.gov/NR/rdonlyres/91D1FA75-C185-491E-A882-49EE17789DF8/0/Manual_071008_Final.pdf
- County of Los Angeles LID Standards Manual: http://dpw.lacounty.gov/wmd/LA_County_LID_Manual.pdf
- SMC LID Manual (pp 114): http://www.lowimpactdevelopment.org/guest75/pub/All_Projects/SoCal_LID_Manual/SoCalLID_Manual_FINAL_040910.pdf
- San Diego County LID Handbook Appendix 4 (Factsheet 26): http://www.sdcounty.ca.gov/dplu/docs/LID-Appendices.pdf

BIO-1: Bioretention with Underdrain

Bioretention stormwater treatment facilities are landscaped shallow depressions that capture and filter stormwater runoff. These facilities function as a soil and plant based filtration device that removes pollutants through a variety of physical, biological, and chemical treatment processes. The facilities normally consist of a ponding area, mulch layer, planting soils, and plantings. As stormwater passes down through the planting soil, pollutants are filtered, adsorbed, and biodegraded by the soil and plants. Bioretention with an underdrain is a treatment control measures that can be used for areas with low permeability native soils or steep slopes. Bioretention may be designed without an underdrain to serve as a retention BMP in areas of high soil permeability (see INF-3 Bioretention) or partial retention/ partial biofiltration BMP (see INF-7: Bioinfiltration).

Bioretention in Parking Lots

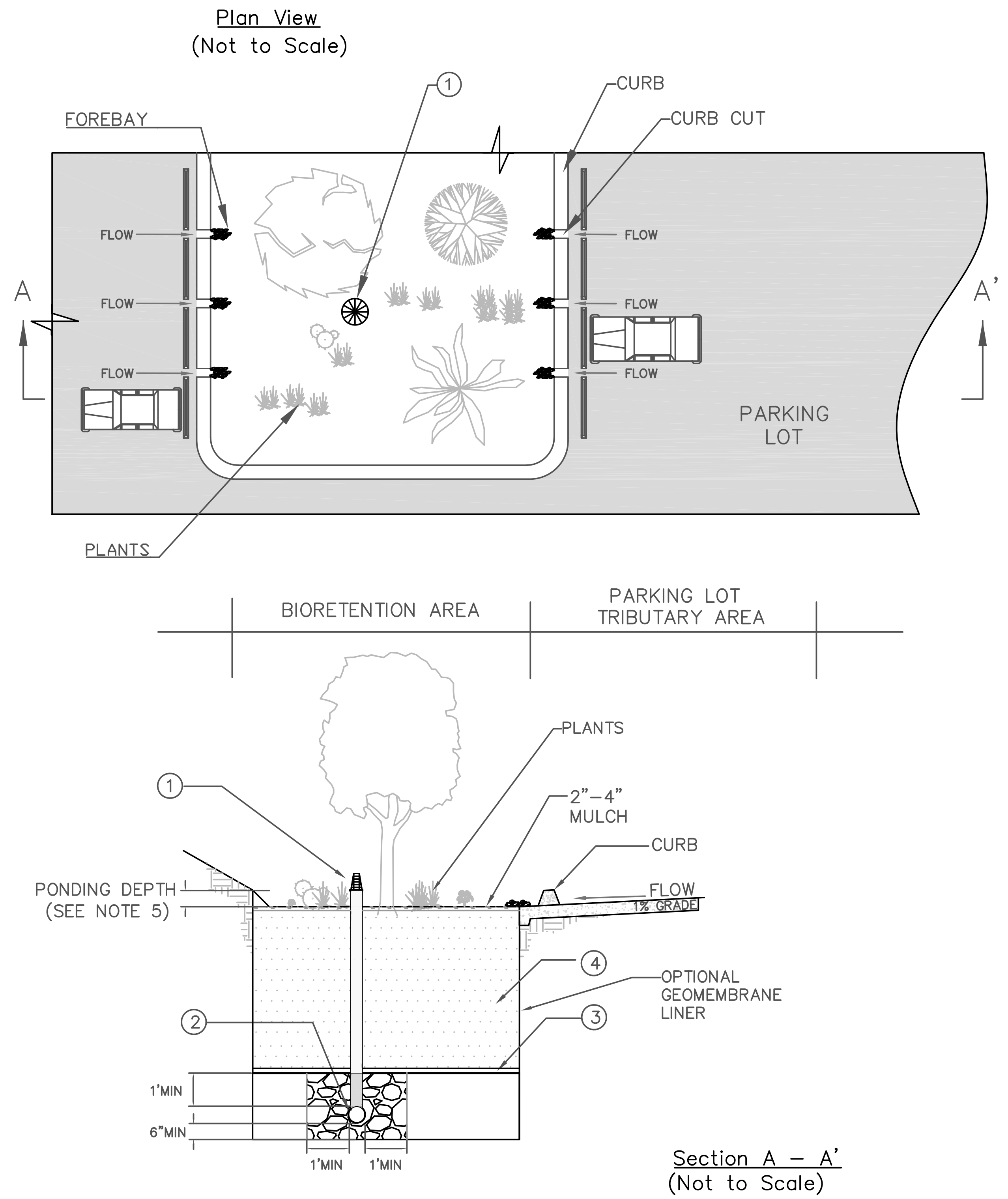
Photo Credits: Geosyntec Consultants

Application

- Parking lots
- Roadway parkways and medians
- School entrances, courtyards, and walkways
- Playgrounds and sports fields

Preventative Maintenance

- Repair small eroded areas
- Remove trash and debris and rake surface soils
- Remove accumulated fine sediments, dead leaves, and trash
- Remove weeds and prune back excess plant growth
- Remove sediment and debris accumulation near inlet and outlet structures
- Periodically observe function under wet weather conditions



NOTES:

1. OVERFLOW DEVICE: VERTICAL RISER OR EQUIVALENT.
2. PERFORATED 6" MIN PVC PIPE UNDERDRAIN.
3. OPTIONAL CHOKING GRAVEL LAYER.
4. 2' MIN PLANTING MIX; 3' PREFERRED.
5. PONDING DEPTH 18" WITH FENCE; 6" WITHOUT FENCE.

Geosyntec consultants

Figure 6-10: Bioretention with Underdrain

Limitations

1) Vertical relief and proximity to storm drain - site must have adequate relief between land surface and storm drain to permit vertical percolation through the soil media and collection and conveyance in underdrain to storm drain system.

2) Depth to groundwater - shallow groundwater table may not permit complete drawdown between storms.

Design Criteria

Bioretention with an underdrain should be designed according to the requirements listed in Table 6-18 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-18: Bioretention with an Underdrain Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume (SQDV)	acre-feet	See Section 2 and Appendix E for calculating SQDV.
Forebay	-	Forebay should be provided for all tributary surfaces that contain landscaped areas. Forebays should be designed to prevent standing water during dry weather and should be planted with a plant palette that is tolerant of wet conditions.
Maximum drawdown time of water ponded on surface	hours	72
Maximum drawdown time of surface ponding plus subsurface pores	hours	96 (72 preferred)
Maximum ponding depth	inches	18 inches
Minimum thickness of amended soils layer	feet	2 (3 preferred)
Minimum thickness of stabilized mulch	inches	2 to 4
Planting mix composition	-	60 to 80% fine sand, 20 to 40% compost
Underdrain sizing	-	6 inch minimum diameter; 0.5% minimum slope; slotted, polyvinyl chloride (PVC) pipe (PVC SDR 35 or approved equivalent); spacing shall be determined to provide

Design Parameter	Unit	Design Criteria
		capacity for maximum rate filtered through amended media
Gravel layer	-	A gravel bed should be provided around underdrain. Underdrain should have at least 1 foot of gravel installed to the sides and on top of the underdrain, and at least 0.5 feet of gravel installed below underdrain.
Overflow device	-	Required

Sizing Criteria

Bioretention facilities with underdrains shall be designed to capture and treat the SQDV. However because these systems commonly have a relatively high amended soil infiltration rate and shallow depth, these systems are typically capable of filtering a significant portion of the SQDV during a storm event. Therefore, a simplified routing approach is described in the following steps that accounts for the portion of the SQDV that is filtered during the storm event.

Step 1: Calculate the Design Volume

Bioretention facilities shall be sized to capture and biofilter the SQDV (see Section 2.3 and Appendix E).

Step 2: Determine the Design Percolation Rate

Sizing is based on the design saturated hydraulic conductivity (K_{sat}) of the amended soil layer. A target K_{sat} of 5 inches per hour is recommended for non-proprietary amended soil media. The media K_{sat} will decline between maintenance cycles as the surface becomes occluded and particulates accumulate in the amended soil layer. A factor of safety of 2.0 should be applied such that the resulting recommended design K_{sat} is 2.5 inches per hour. This value should be used for sizing unless sufficient rationale is provided to justify a higher design K_{sat}.

Step 3: Calculate the surface area

Determine the size of the required infiltrating surface by assuming the SQDV will fill the available ponding depth plus the void spaces in the media, based on the computed porosity of the filter media and aggregate layer.

1) Select a surface ponding depth (d_p) that satisfies geometric criteria and is congruent with the constraints of the site. Selecting a deeper ponding depth (18 inches maximum) generally yields a smaller footprint, however, it requires greater consideration for public safety, energy dissipation, and plant selection.

2) Compute time for selected ponding depth to filter through media:

$$t_{ponding} = \frac{d_p}{K_{design}} 12 \frac{in}{ft} \qquad \text{(Equation 6-24)}$$

Where:

$t_{ponding}$ = required drain time of surface ponding (≤ 72 hrs)

d_p = selected surface ponding water depth (ft)

K_{design} = media design saturated hydraulic conductivity (in/hr) (see Step 2, above)

If $t_{ponding}$ exceeds 72 hours, return to (1) and reduce surface ponding or increase media K_{design}. Otherwise, proceed to next step.

Note: In nearly all cases, $t_{ponding}$ will not approach 72 hours unless a low K_{design} is specified.

3) Compute depth of water that may be filtered during the design storm event as follows:

$$d_{filtered} = Minimum\left[\frac{K_{design} \times T_{routing}}{12 \, in/ft}, d_p\right] \qquad \text{(Equation 6-25)}$$

Where:

$d_{filtered}$ = depth of water that may be considered to be filtered during the design storm event (ft) for routing calculations; this value should not exceed the surface ponding depth (d_p)

K_{design} = design saturated hydraulic conductivity (in/hr) (see Step 2, above)

$T_{routing}$ = storm duration that may be assumed for routing calculations; this should be assumed to be 3 hours unless rationale for an alternative assumption is provided

d_p = selected surface ponding water depth (ft)

The intention is that routing is important in the appropriate sizing of bioretention with underdrains. However, the depth of water considered to be filtered during the storm should be limited to the maximum ponding depth. This *results in designs that are robust to account for a variety of storm depths and durations. This limitation is for sizing calculations only. In reality, the depth that is filtered during a storm will vary based on storm depth, duration, and intensity. This TGM does not intend to limit the amount that may actually be filtered.*

4) Calculate required infiltrating surface area (filter bottom area):

$$A_{req} = \frac{SQDV}{d_p + d_{filtered}} \qquad \text{(Equation 6-26)}$$

Where:

A_{req}	=	required infiltrating area (ft^2). Should be calculated at the contour corresponding to the mid ponding depth (i.e., $0.5 \times d_p$ from the bottom of the facility)
SQDV	=	stormwater quality design volume (ft^3)
d_p	=	selected surface ponding water depth (ft)
$d_{filtered}$	=	depth of water that can be considered to be filtered during the design storm event (ft) for routing calculations (See Equation 6-15)

5) Calculate total footprint required by including a buffer for side slopes and freeboard; A_{req} is calculated at the contour corresponding to the mid ponding depth (i.e., $0.5 \times d_p$ from the bottom of the facility).

Geometry

1) Minimum planting soil depth should be 2 feet, although 3 feet is preferred.

   The intention is that the minimum planting soil depth should provide a beneficial root zone for the chosen plant palette and adequate water storage for the stormwater quality design volume. A deeper soil depth will provide a smaller surface area footprint.

2) Bioretention should be designed to drain below the planting soil in less than 72 hours and completely drain from the underdrain in 96 hours (both starting from the end of inflow).

   The intention is that soils must be allowed to dry out periodically in order to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates, maintain adequate soil oxygen levels for healthy soil biota and vegetation, and to provide proper soil conditions for biodegradation and retention of pollutants.

Flow Entrance and Energy Dissipation

The following types of flow entrance can be used for bioretention cells:

1) Dispersed, low velocity flow across a landscape area. Dispersed flow may not be possible given space limitations or if the facility is controlling roadway or parking lot flows where curbs are mandatory.

2) Dispersed flow across pavement or gravel and past wheel stops for parking areas.

3) Curb cuts for roadside or parking lot areas: Curb cuts should include rock or other erosion protection material in the channel entrance to dissipate energy. Flow entrance should drop 2 to 3 inches from curb line and provide an area for settling and periodic removal of sediment and coarse material before flow dissipates to the remainder of the cell.

4) Pipe flow entrance: Piped entrances, such as roof downspouts, should include rock, splash blocks, or other appropriate measures at the entrance to dissipate energy and disperse flows.

5) Woody plants (trees, shrubs, etc.) can restrict or concentrate flows and can be damaged by erosion around the root ball and should not be placed directly in the entrance flow path.

Underdrains

Underdrains should meet the following criteria:

1) 6-inch minimum diameter.

2) Underdrains should be made of slotted, polyvinyl chloride (PVC) pipe (PVC SDR 35 or approved equivalent). *The intention is that compared to round-hole perforated pipe, slotted underdrains provide greater intake capacity, clog resistant drainage, and reduced entrance velocity into the pipe, thereby reducing the chances of solids migration.*

3) Slotted pipe should have 2 to 4 rows of slots cut perpendicular to the axis of the pipe or at right angles to the pitch of corrugations. Slots should be 0.04 to 0.1 inches and should have a length of 1 to 1.25 inches. Slots should be longitudinally spaced such that the pipe has a minimum of one square inch of slot per lineal foot of pipe and should be placed with slots facing the bottom of the pipe.

4) Underdrains should be sloped at a minimum of 0.5%.

5) Rigid non-perforated observation pipes with a diameter equal to the underdrain diameter should be connected to the underdrain every 100 feet to provide a clean-out port as well as an observation well to monitor dewatering rates. The wells/cleanouts should be connected to the perforated underdrain with the appropriate manufactured connections. The wells/cleanouts should extend 6 inches above the top

elevation of the bioretention facility mulch, and should be capped with a lockable screw cap. The ends of the underdrain pipes not terminating in an observation well/cleanout should also be capped.

6) The following aggregate should be used to provide a gravel blanket and bedding for the underdrain pipe. Place the underdrain on a bed of washed aggregate at a minimum thickness of 6 inches and cover it with the same aggregate to provide a 1 foot minimum depth around the top and sides of the slotted pipe.

Sieve size	Percent Passing
¾ inch	100
¼ inch	30-60
US No. 8	20-50
US No. 50	3-12
US No. 200	0-1

7) At the option of the designer/geotechnical engineer, a geotextile fabric may be placed between the planting media and the drain rock. If a geotextile fabric is used, it should meet a minimum permittivity rate of 75 gal/min/ft^2, should not impede the infiltration rate of the soil medium, and should meet the following minimum materials requirements.

Geotextile Property	Value	Test Method
Trapezoidal Tear (lbs)	40 (min)	ASTM D4533
Permeability (cm/sec)	0.2 (min)	ASTM D4491
AOS (sieve size)	#60 - #70 (min)	ASTM D4751
Ultraviolet resistance	70% or greater	ASTM D4355

Preferably, aggregate should be used in place of filter fabric to reduce the potential for clogging. This aggregate layer should consist of 2 to 4 inches of washed sand underlain with 2 inches of choking stone (Typically #8 or #89 washed).

8) For bioretention facilities enhanced to remove address nitrogen as the primary pollutant class, the underdrain should be elevated from the bottom of the bioretention facility by at least 6 inches within the gravel blanket to create a fluctuating anaerobic/aerobic zone below the drain pipe. *The intention is that denitrification within the anaerobic/anoxic zone is facilitated by microbes using forms of nitrogen (NO_2 and NO_3) instead of oxygen for respiration.*

An alternative enhanced nitrogen removal design is to include an internal water storage layer by adding a 90-degree elbow to the underdrain to raise the outlet. This design feature provides additional storage in the media. The bioretention facility must have at least 30 inches of planting media. The top of the elbow should be at least 12 inches below the top of the planting media, and in poorly draining soils, should preferably be 18 to 24 inches below the top of the planting media. The top of the water storage layer should not be less than 12 inches from the bottom of the planting media layer. (For more information, see Urban Waterways publication).

9) The underdrain should drain freely to an acceptable discharge point. The underdrain can be connected to a downstream open conveyance (vegetated swale), to another bioretention cell as part of a connected treatment system, to a storm drain, daylight to a vegetated dispersion area using an effective flow dispersion device, or to a storage facility for rainwater harvesting.

Overflow

An overflow device is required at the maximum ponding depth. The following, or equivalent, should be provided:

1) A vertical PVC pipe (SDR 35) should be connected to the underdrain.
2) The overflow riser(s) should be 6 inches or greater in diameter, so it can be cleaned without damage to the pipe. The vertical pipe will provide access to cleaning the underdrains.
3) The inlet to the riser should be at the ponding depth (maximum 18 inches for fenced bioretention areas and 6 inches for areas that are not fenced), and be capped with a spider cap to exclude floating mulch and debris. Spider caps should be screwed in or glued (i.e., not removable).

Hydraulic Restriction Layers

Infiltration pathways may need to be restricted due to the close proximity of roads, foundations, or other infrastructure. A geomembrane liner, or other equivalent water proofing, may be placed along the vertical walls to reduce lateral flows. This liner should have a minimum thickness of 30 mils.

Planting/Storage Media

1) The planting media placed in the cell should achieve a long-term, in-place infiltration rate of at least 1 inch per hour. Higher infiltration rates are permissible. If the design long-term, in-place infiltration rate of the soil exceeds 12 inches per hour, documentation should be provided to demonstrate that the media will adequately address pollutants of concern at a higher flowrate. Bioretention soil shall also support vigorous plant growth.
2) Planting media should consist of 60 to 80% fine sand and 20 to 40% compost.
3) Sand should be free of wood, waste, coating such as clay, stone dust, carbonate, etc., or any other deleterious material. All aggregate passing the No. 200 sieve size should be non-plastic. Sand for bioretention should be analyzed by an accredited lab using

#200, #100, #40, #30, #16, #8, #4, and 3/8 sieves (ASTM D 422 or as approved by the local permitting authority) and meet the following gradation (Note: all sands complying with ASTM C33 for fine aggregate comply with the gradation requirements below):

Sieve Size (ASTM D422)	% Passing (by weight)	
	Minimum	Maximum
3/8 inch	100	100
#4	90	100
#8	70	100
#16	40	95
#30	15	70
#40	5	55
#100	0	15
#200	0	5

Note: the gradation of the sand component of the media is believed to be a major factor in the hydraulic conductivity of the media mix. If the desired hydraulic conductivity of the media cannot be achieved within the specified proportions of sand and compost (#2), then it may be necessary to utilize sand at the coarser end of the range specified in above ("minimum" column).

4) Compost should be a well decomposed, stable, weed free organic matter source derived from waste materials including yard debris, wood wastes, or other organic materials not including manure or biosolids meeting standards developed by the US Composting Council (USCC). The product shall be certified through the USCC Seal of Testing Assurance (STA) Program (a compost testing and information disclosure program). Compost quality should be verified via a lab analysis to be:

- Feedstock materials shall be specified and include one or more of the following: landscape/yard trimmings, grass clippings, food scraps, and agricultural crop residues.
- Organic matter: 35-75% dry weight basis.
- Carbon and Nitrogen Ratio: 15:1 < C:N < 25:1
- Maturity/Stability: shall have dark brown color and a soil-like odor. Compost exhibiting a sour or putrid smell, containing recognizable grass or leaves, or is hot (120 F) upon delivery or rewetting is not acceptable.
- Toxicity: any one of the following measures is sufficient to indicate non-toxicity:
    - NH4:NH3 < 3
    - Ammonium < 500 ppm, dry weight basis

  - Seed Germination > 80% of control
  - Plant trials > 80% of control
  - Solvita® > 5 index value
- Nutrient content:
  - Total Nitrogen content 0.9% or above preferred
  - Total Boron should be <80 ppm, soluble boron < 2.5 ppm
- Salinity: < 6.0 mmhos/cm
- pH between 6.5 and 8 (may vary with plant palette)

Compost for bioretention should be analyzed by an accredited lab using #200, ¼ inch, ½ inch, and 1 inch sieves (ASTM D 422 or as approved by the local permitting authority) and meet the following gradation:

Sieve Size (ASTM D422)	% Passing (by weight)	
	Minimum	Maximum
1 inch	99	100
½ inch	90	100
¼ inch	40	90
#200	2	10

Tests should be sufficiently recent to represent the actual material that is anticipated to be delivered to the site. If processes or sources used by the supplier have changed significantly since the most recent testing, new tests should be requested.

Note: the gradation of compost used in bioretention media is believed to play an important role in the saturated hydraulic conductivity of the media. To achieve a higher saturated hydraulic conductivity, it may be necessary to utilize compost at the coarser end of this range ("minimum" column). The percent passing the #200 sieve (fines) is believed to be the most important factor in hydraulic conductivity.

In addition, a coarser compost mix provides more heterogeneity of the bioretention media, which is believed to be advantageous for more rapid development of soil structure needed to support health biological processes. This may be an advantage for plant establishment with lower nutrient and water input.

5) The bioretention area should be covered with 2 to 4 inches (average 3 inches) of mulch at the start and an additional placement of 1 to 2 inches of mulch should be added annually. *The intention is that to help sustain the nutrient levels, suppress weeds, retain moisture, and maintain infiltration capacity*.

Plants

Plant materials should be tolerant of summer drought, ponding fluctuations, and saturated soil conditions for 48 to 96 hours.

It is recommended that a minimum of three types of tree, shrubs, and/or herbaceous groundcover species be incorporated to protect against facility failure due to disease and insect infestations of a single species.

Native plant species and/or hardy cultivars that are not invasive and do not require chemical inputs should be used to the maximum extent practicable.

Operations and Maintenance

Bioretention areas require annual plant, soil, and mulch layer maintenance to ensure optimum infiltration, storage, and pollutant removal capabilities. In general, bioretention maintenance requirements are typical landscape care procedures and include:

1) Watering: Plants should be selected to be drought-tolerant and not require watering after establishment (2 to 3 years). Watering may be required during prolonged dry periods after plants are established.

2) Erosion control: Inspect flow entrances, ponding area, and surface overflow areas periodically, and replace soil, plant material, and/or mulch layer in areas if erosion has occurred (see Appendix I for a bioretention inspection and maintenance checklist). Properly designed facilities with appropriate flow velocities should not have erosion problems except perhaps in extreme events. If erosion problems occur, the following should be reassessed: (1) flow velocities and gradients within the cell, and (2) flow dissipation and erosion protection strategies in the pretreatment area and flow entrance. If sediment is deposited in the bioretention area, immediately determine the source within the contributing area, stabilize, and remove excess surface deposits.

3) Plant material: Depending on aesthetic requirements, occasional pruning and removing of dead plant material may be necessary. Replace all dead plants and if specific plants have a high mortality rate, assess the cause and, if necessary, replace with more appropriate species. Periodic weeding is necessary until plants are established. The weeding schedule should become less frequent if the appropriate plant species and planting density have been used and, as a result, undesirable plants have been excluded.

4) Nutrient and pesticides: The soil mix and plants are selected for optimum fertility, plant establishment, and growth. Nutrient and pesticide inputs should not be required and may degrade the pollutant processing capability of the bioretention area, as well as contribute pollutant loads to receiving waters. By design, bioretention facilities are located in areas where phosphorous and nitrogen levels are often

elevated and these should not be limiting nutrients. If in question, have soil analyzed for fertility.

5) Mulch: Replace mulch annually in bioretention facilities where high trash, sediment load, and heavy metal deposition is likely (e.g., heavy metal contributing areas include industrial and auto dealer/repair parking lots and roads). In residential lots or other areas where metal deposition is not a concern, replace or add mulch as needed to maintain a 2 to 3 inch depth at least once every two years.

6) Soil: Soil mixes for bioretention facilities are designed to maintain long-term fertility and pollutant processing capability. Replacing mulch in bioretention facilities where high trash, sediment load, and heavy metal deposition are likely provides an additional level of protection for prolonged performance. Estimates from metal attenuation research suggest that metal accumulation should not present an environmental concern for at least 20 years in bioretention systems. However, the saturated hydraulic conductivity should be assessed at least annually to ensure that the design water quality event is being treated. If in question, have soil analyzed for fertility and pollutant levels.

BIO-2: Planter Box

Planter boxes are bioretention treatment control measures that are completely contained within an impermeable structure with an underdrain (they do not infiltrate). These facilities function as a soil and plant based filtration device that removes pollutants through a variety of physical, biological, and chemical treatment processes. The facilities normally consist of a ponding area, mulch layer, planting soils, plantings, and an underdrain within the planter box. As stormwater passes down through the planting soil, pollutants are filtered, adsorbed, and biodegraded by the soil and plants. Planter boxes are comprised of a variety of materials, usually chosen to be the same material as the adjacent building or sidewalk.

Planter boxes may be placed adjacent to or near buildings, other structures, or sidewalks. Planter boxes can be used directly adjacent to buildings beneath downspouts as long as the boxes are properly lined on the building side and the overflow outlet discharges away from the building to ensure water does not percolate into footings or foundations. They can also be placed further away from buildings by conveying roof runoff in shallow engineered open conveyances, shallow pipes, or other innovative drainage structures.

Planter boxes extending along a building wall

Photo Credit: Geosyntec Consultants

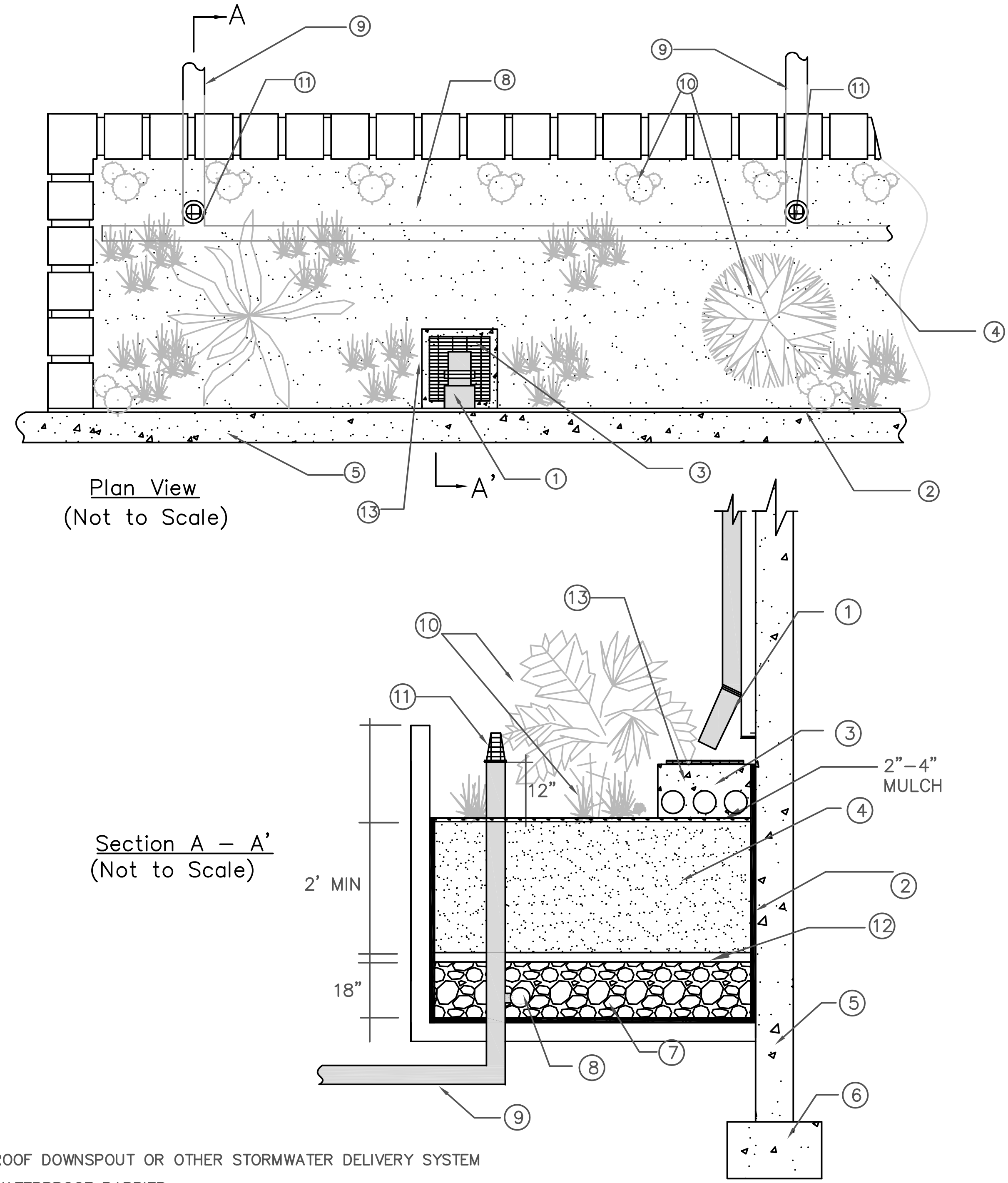
<u>Application</u>

- Areas adjacent to buildings and sidewalks
- Building entrances, courtyards, and walkways

<u>Preventative Maintenance</u>

- Repair small eroded areas
- Remove trash and debris and rake surface soils
- Remove accumulated fine sediments, dead leaves, and trash
- Remove weeds and prune back excess plant growth
- Remove sediment and debris accumulation near inlet and outlet structures

  Periodically observe function under wet weather conditions



NOTES:

1. ROOF DOWNSPOUT OR OTHER STORMWATER DELIVERY SYSTEM
2. WATERPROOF BARRIER
3. SHALLOW ENERGY DISSIPATOR BASIN DISPERSES FLOW AT SOIL SURFACE
4. SOIL MIX (SEE PLANTING MEDIA SECTION)
5. BUILDING
6. FOUNDATION. INSTALL FOUNDATION DRAINS AS NEEDED
7. GRAVEL BEDDING (SEE UNDERDRAIN)
8. PERFORATED PIPE SHALL RUN ENTIRE LENGTH OF PLANTER
9. CONNECTION TO DOWNSTREAM CONVEYANCE SYSTEM
10. PLANTS
11. SET OVERFLOW 2" BELOW THE TOP OF THE PLANTER
12. OPTIONAL CHOKING GRAVEL LAYER

Geosyntec consultants

Figure 6-11: Planter Box

Limitations

The applicability of stormwater planter boxes is limited by the following site characteristics:

1) The tributary area (area draining to the planter box area) should be less than 15,000 ft^2.

2) Groundwater levels should be at least 2 ft lower than the bottom of the planter box.

3) Site must have adequate vertical relief between land surface and the stormwater conveyance system to permit connection of the underdrain to the stormwater conveyance system.

4) Planter boxes should not be located in areas with excessive shade to avoid poor vegetative growth. For moderately shaded areas, shade tolerant plants should be used.

Design Criteria

Planter boxes should be designed according to the requirements listed in Table 6-19 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-19: Planter Box Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume (SQDV)	acre-feet	See Section 2 and Appendix E for calculating SQDV.
Drawdown time of planting soil	hours	12
Maximum ponding depth	inches	12
Minimum soil depth	feet	2; 3 preferred
Stabilized mulch depth	inches	2 to 3
Planting soil composition	-	60 to 70% sand, 30 to 40% compost
Underdrain	-	6 inch minimum diameter; 0.5% minimum slope; slotted, polyvinyl chloride (PVC) pipe (PVC SDR 35 or approved equivalent)
Overflow device	-	Required

Sizing Criteria

See Sizing Criteria section in the BIO-1: Bioretention with underdrains fact sheet.

Geometry and Size

1) Planter boxes areas should be sized to capture and treat the SQDV with a 12 inch maximum ponding depth. The mulch layer should be included as part of the ponding depth.

2) Minimum soil depth should be 2 feet, although 3 feet is preferred. *The intention is that a minimum soil depth should provide a beneficial root zone for the chosen plant palette and adequate water storage for the SQDV. A deeper planting soil depth will provide a smaller surface area footprint.*

3) Planter boxes should be designed to drain to below the planting soil depth in less than 48 hours. *The intention is that soils must be allowed to dry out periodically in order to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates, prevent long periods of saturation for plant health, maintain adequate soil oxygen levels for healthy soil biota and vegetation, reduce potential for vector breeding, and provide proper soil conditions for biodegradation and retention of pollutants.*

4) Any planter box shape configuration is possible as long as other design criteria are met.

5) The distance between the downspouts and the overflow outlet should be maximized. *The intention is to increase the opportunity for stormwater retention and filtration.*

6) Off-line configurations should be considered to minimize the possibility of scouring and resuspension of previously captured pollutants during large storms.

Structural Materials

1) Planter boxes should be constructed out of stone, concrete, brick, recycled plastic, or other permanent materials. Pressure-treated wood or other materials that may leach pollutants (e.g., arsenic, copper, zinc, etc.) should not be allowed.

2) The structure should be adequately sealed or a waterproof membrane installed to ensure water only exits the structure via the underdrain.

Flow Entrance and Energy Dissipation

The following types of flow entrance can be used for planter boxes:

1) Pipe flow entrance: Piped entrances, such as roof downspouts, should include rock, splash blocks, or other appropriate measures at the entrance to dissipate energy and disperse flows.

2) Woody plants (e.g., trees, shrubs, etc.) can restrict or concentrate flows and can be damaged by erosion around the root ball and should not be placed directly in the entrance flow path.

Underdrains

Underdrains are required and should meet the following criteria:

1) 6-inch minimum diameter.

2) Underdrains should be made of slotted, polyvinyl chloride (PVC) pipe (PVC SDR 35 or approved equivalent). *The intention is that in comparison to round-hole perforated pipe, slotted underdrains provide greater intake capacity, clog resistant drainage, and reduced entrance velocity into the pipe, thereby reducing the chances of solids migration.*

3) Slotted pipe should have 2 to 4 rows of slots cut perpendicular to the axis of the pipe or at right angles to the pitch of corrugations. Slots should be 0.04 to 0.1 inch and should have a length of 1 to 1.25 inches. Slots should be longitudinally spaced such that the pipe has a minimum of one square inch opening per lineal foot and should face down.

4) Underdrains should be sloped at a minimum of 0.5%.

5) Rigid non-perforated observation pipes with a diameter equal to the underdrain diameter should be connected to the underdrain every 100 feet to provide a clean-out port as well as an observation well to monitor dewatering rates. The wells/cleanouts should be connected to the perforated underdrain with the appropriate manufactured connections. The wells/cleanouts should extend 6 inches above the top elevation of the bioretention facility mulch, and should be capped with a lockable screw cap. The ends of underdrain pipes not terminating in an observation well/cleanout should also be capped.

6) The following aggregate should be used to provide a gravel blanket and bedding for the underdrain pipe. Place the underdrain on a bed of washed aggregate at a minimum thickness of 6 inches and cover it with the same aggregate to provide a 1 foot minimum depth around the top and sides of the slotted pipe.

Sieve size	Percent Passing
¾ inch	100
¼ inch	30-60
US No. 8	20-50
US No. 50	3-12
US No. 200	0-1

7) At the option of the designer/geotechnical engineer, a geotextile fabric may be placed between the planting media and the drain rock. If a geotextile fabric is used, it should

meet a minimum permittivity rate of 75 gal/min/ft^2, should not impede the infiltration rate of the soil medium, and should meet the following minimum materials requirements.

Geotextile Property	Value	Test Method
Trapezoidal Tear (lbs)	40 (min)	ASTM D4533
Permeability (cm/sec)	0.2 (min)	ASTM D4491
AOS (sieve size)	#60 - #70 (min)	ASTM D4751
Ultraviolet resistance	70% or greater	ASTM D4355

Preferably, aggregate should be used in place of filter fabric to reduce the potential for clogging. This aggregate layer should consist of 2 to 4 inches of washed sand underlain with 2 inches of choking stone (Typically #8 or #89 washed).

8) The underdrain should be elevated from the bottom of the bioretention facility by 6 inches within the gravel blanket to create a fluctuating anaerobic/aerobic zone below the drain pipe. *The intention is that denitrification within the anaerobic/anoxic zone is facilitated by microbes using forms of nitrogen (NO_2 and NO_3) instead of oxygen for respiration.*

9) The underdrain must drain freely to an acceptable discharge point. The underdrain can be connected to a downstream open conveyance (vegetated swale), to another bioretention cell as part of a connected treatment system, to a storm drain, daylight to a vegetated dispersion area using an effective flow dispersion device, or to a storage facility for rainwater harvesting.

Overflow

An overflow device is required to be set at 2 inches below the top of the planter and no more than 12 inches above the soil surface. The most common option is a vertical riser, described below.

Vertical riser

1) A vertical PVC pipe (SDR 35) should be connected to the underdrain.

2) The overflow riser(s) should be 6 inches or greater in diameter, so it can be cleaned without damage to the pipe. The vertical pipe will provide access to cleaning the underdrains.

3) The inlet to the riser should be a maximum of 12 inches above the planting soil, and be capped with a spider cap. Spider caps should be screwed in or glued (i.e., not removable).

Hydraulic Restriction Layers

A waterproof barrier should be provided to restrict moisture away from foundations. Geomembrane liners should have a minimum thickness of 30 mils. Equivalent waterproofing measures may be used.

Planting/Storage Media

1) The planting media placed in the cell should achieve a long-term, in-place infiltration rate of at least 1 inch per hour. Higher infiltration rates are permissible. If the design long-term, in-place infiltration rate of the soil exceeds 12 inches per hour, documentation should be provided to demonstrate that the media will adequately address pollutants of concern at a higher flowrate. Planter box soil shall also support vigorous plant growth.

2) Planting media should consist of 60 to 80% fine sand and 20 to 40% compost.

3) Sand should be free of wood, waste, coating such as clay, stone dust, carbonate, etc., or any other deleterious material. All aggregate passing the No. 200 sieve size should be non-plastic. Sand for the planter box should be analyzed by an accredited lab using #200, #100, #40, #30, #16, #8, #4, and 3/8 sieves (ASTM D 422 or as approved by the local permitting authority) and meet the following gradation (Note: all sands complying with ASTM C33 for fine aggregate comply with the gradation requirements below):

Sieve Size (ASTM D422)	% Passing (by weight)	
	Minimum	Maximum
3/8 inch	100	100
#4	90	100
#8	70	100
#16	40	95
#30	15	70
#40	5	55
#100	0	15
#200	0	5

Note: the gradation of the sand component of the media is believed to be a major factor in the hydraulic conductivity of the media mix. If the desired hydraulic conductivity of the media cannot be achieved within the specified proportions of sand and compost (#2), then it may be necessary to utilize sand at the coarser end of the range specified in above (“minimum” column).

4) Compost should be a well decomposed, stable, weed free organic matter source derived from waste materials including yard debris, wood wastes, or other organic materials not including manure or biosolids meeting standards developed by the US Composting Council (USCC). The product shall be certified through the USCC Seal

of Testing Assurance (STA) Program (a compost testing and information disclosure program). Compost quality should be verified via a lab analysis to be:

- Feedstock materials shall be specified and include one or more of the following: landscape/yard trimmings, grass clippings, food scraps, and agricultural crop residues.
- Organic matter: 35-75% dry weight basis.
- Carbon and Nitrogen Ratio: 15:1 < C:N < 25:1
- Maturity/Stability: shall have dark brown color and a soil-like odor. Compost exhibiting a sour or putrid smell, containing recognizable grass or leaves, or is hot (120 F) upon delivery or rewetting is not acceptable.
- Toxicity: any one of the following measures is sufficient to indicate non-toxicity:
    - $NH_4:NH_3 < 3$
    - Ammonium < 500 ppm, dry weight basis
    - Seed Germination > 80% of control
    - Plant trials > 80% of control
    - Solvita® > 5 index value
- Nutrient content:
    - Total Nitrogen content 0.9% or above preferred
    - Total Boron should be <80 ppm, soluble boron < 2.5 ppm
- Salinity: < 6.0 mmhos/cm
- pH between 6.5 and 8 (may vary with plant palette)

Compost for planter box should be analyzed by an accredited lab using #200, ¼ inch, ½ inch, and 1 inch sieves (ASTM D 422 or as approved by the local permitting authority) and meet the following gradation:

Sieve Size (ASTM D422)	% Passing (by weight)	
	Minimum	Maximum
1 inch	99	100
½ inch	90	100
¼ inch	40	90
#200	2	10

Tests should be sufficiently recent to represent the actual material that is anticipated to be delivered to the site. If processes or sources used by the supplier have changed significantly since the most recent testing, new tests should be requested.

Note: the gradation of compost used in planter box media is believed to play an important role in the saturated hydraulic conductivity of the media. To achieve a higher saturated hydraulic conductivity, it may be necessary to utilize compost at the coarser end of this range ("minimum" column). The percent passing the #200 sieve (fines) is believed to be the most important factor in hydraulic conductivity.

In addition, a coarser compost mix provides more heterogeneity of the planter box media, which is believed to be advantageous for more rapid development of soil structure needed to support health biological processes. This may be an advantage for plant establishment with lower nutrient and water input.

5) The planter box should be covered with 2 to 4 inches (average 3 inches) of mulch at the start and an additional placement of 1 to 2 inches of mulch should be added annually. *The intention is that to help sustain the nutrient levels, suppress weeds, retain moisture, and maintain infiltration capacity*.

Plants

1) Plant materials should be tolerant of summer drought, ponding fluctuations, and saturated soil conditions for 48 to 96 hours.

2) It is recommended that a minimum of three types of tree, shrubs, and/or herbaceous groundcover species be incorporated to protect against facility failure due to disease and insect infestations of a single species.

3) Native plant species and/or hardy cultivars that are not invasive and do not require chemical inputs should be used to the maximum extent practicable.

4) Plants should be selected carefully to minimize maintenance and function properly.

Operations and Maintenance

Planter boxes require annual plant, soil, and mulch layer maintenance to ensure optimum infiltration, storage, and pollutant removal capabilities. In general, planter box maintenance requirements are typical of landscape care procedures and include:

1) Watering: Plants should be selected to be drought-tolerant and do not require watering after establishment (2 to 3 years). Watering may be required during prolonged dry periods after plants are established.

2) Erosion control: Inspect flow entrances, ponding area, and surface overflow areas periodically, and replace soil, plant material, and/or mulch layer in areas if erosion has occurred (see Appendix I for an inspection and maintenance checklist). Properly designed facilities with appropriate flow velocities should not have erosion problems

except perhaps in extreme events. If erosion problems occur, the following should be reassessed: (1) flow velocities and gradients within the cell, and (2) flow dissipation and erosion protection strategies in the flow entrance. If sediment is deposited in the planter box, immediately determine the source within the contributing area, stabilize, and remove excess surface deposits.

3) Plant material: Depending on aesthetic requirements, occasional pruning and removing of dead plant material may be necessary. Replace all dead plants and if specific plants have a high mortality rate, assess the cause and, if necessary, replace with more appropriate species. Periodic weeding is necessary until plants are established. The weeding schedule should become less frequent if the appropriate plant species and planting density have been used and, as a result, undesirable plants have been excluded.

4) Nutrients and pesticides: The soil mix and plants are selected for optimum fertility, plant establishment, and growth. Nutrient and pesticide inputs should not be required and may degrade the pollutant processing capability of the planter box area, as well as contribute pollutant loads to receiving waters. By design, planter boxes are located in areas where phosphorous and nitrogen levels are often elevated and these should not be limiting nutrients. If in question, have soil analyzed for fertility.

5) Mulch: Replace mulch annually in planter boxes where high trash, sediment load, and heavy metal deposition is likely (e.g., heavy metal contributing areas include industrial, auto dealer/repair, parking lots, and roads). In residential lots or other areas where metal deposition is not a concern, replace or add mulch as needed to maintain a 2 to 3 inch depth at least once every two years.

6) Soil: Soil mixes for planter boxes are designed to maintain long-term fertility and pollutant processing capability. Replacing mulch in planter boxes where high trash, sediment load, and heavy metal deposition are likely provides an additional level of protection for prolonged performance. Estimates from metal attenuation research suggest that metal accumulation should not present an environmental concern for at least 20 years in planter boxes. However, the saturated hydraulic conductivity should be assessed at least annually to ensure that the design water quality event is being treated. If in question, have soil analyzed for fertility and pollutant levels.

BIO-3: Vegetated Swale

Vegetated swales are open, shallow channels with low-lying vegetation covering the side slopes and bottom that collect and slowly convey runoff to downstream discharge points. Vegetated swales provide pollutant removal through settling and filtration in the vegetation (usually grasses) lining the channels, provide the opportunity for stormwater volume reduction through infiltration and evapotranspiration, reduce the flow velocity, and conveying stormwater runoff. An effective vegetated swale achieves uniform sheet flow through a densely vegetated area for a period of several minutes. The vegetation in the swale can vary depending on its location and is the choice of the designer, depending on the design criteria outlined in this section.

Vegetated swale captures flow from a residential street

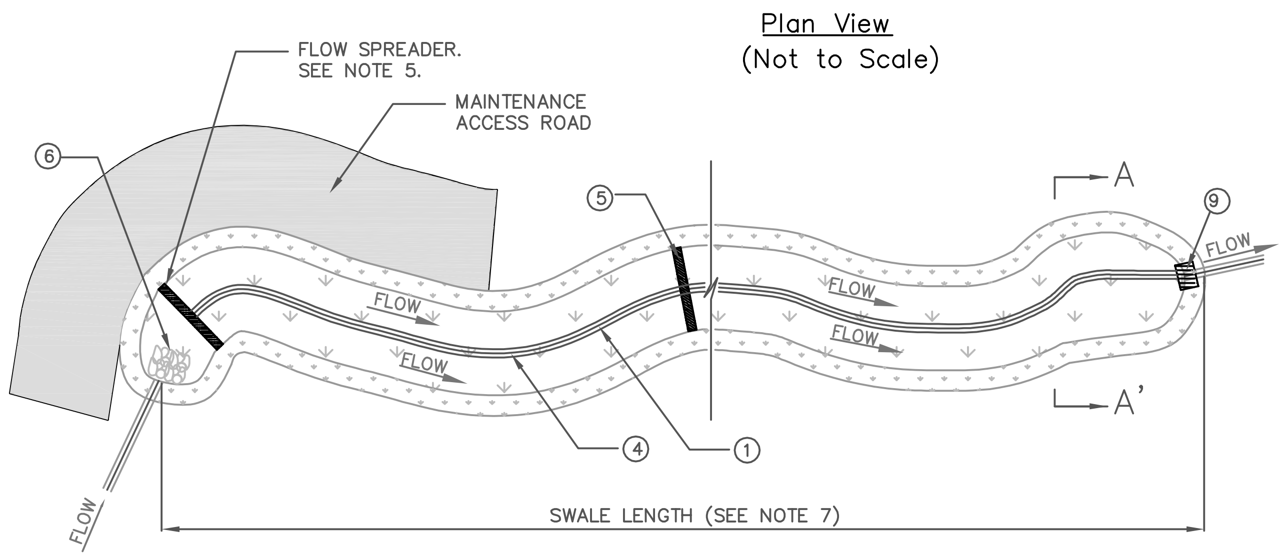
Photo Credit: Geosyntec Consultants

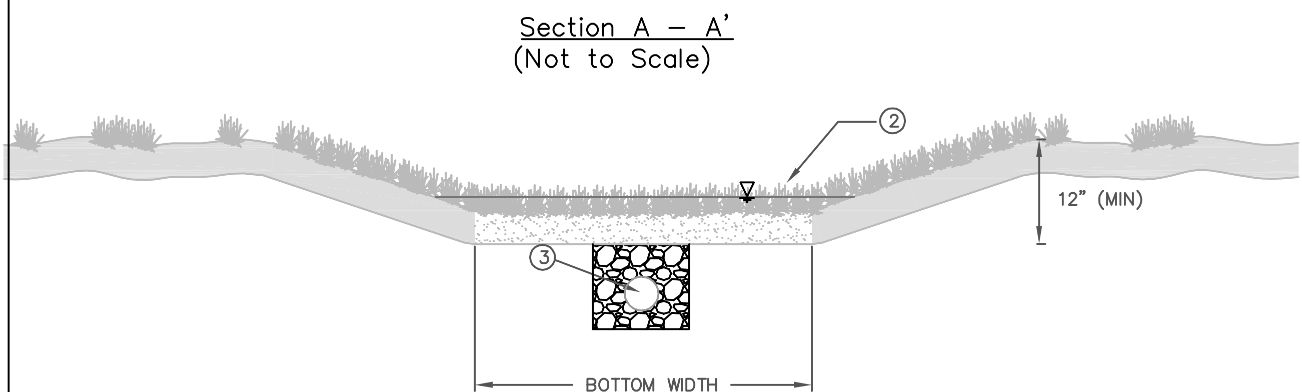
Application

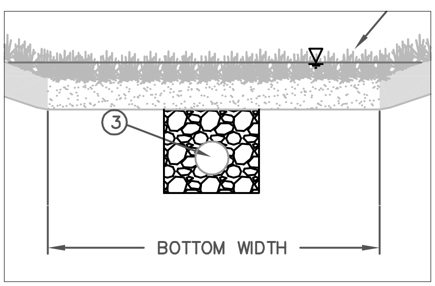
- Open areas adjacent to parking lots
- Open spaces adjacent to athletic fields
- Roadway medians and shoulders

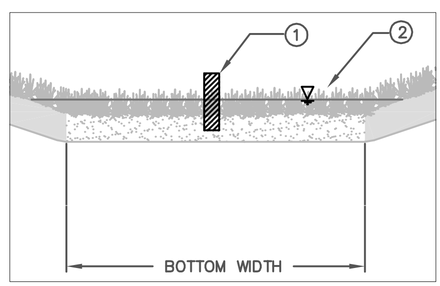
Preventative Maintenance

- Remove excess sediment, trash, and debris
- Clean and reset flow spreaders
- Mow regularly
- Remove sediment and debris build-up near inlets and outlets
- Repair minor erosion and scouring









NOTES:

1. SWALE DIVIDER REQUIRED FOR BOTTOM WIDTHS > 10'. MINIMUM REQUIRED BOTTOM WIDTH IS 2' EXCLUDING WIDTH OF LOW FLOW CHANNEL. MAXIMUM BOTTOM WIDTH WITH DIVIDER IS 16'.
2. DEPTH OF FLOW FOR WATER QUALITY TREATMENT MUST NOT EXCEED TWO-THIRDS OF VEGETATION HEIGHT OR NOT GREATER THAN 2" FOR FREQUENTLY MOWED TURF.
3. IF AN UNDERDRAIN IS REQUIRED, IT MUST CONSIST OF AN AT LEAST 6" DIAMETER PERFORATED PIPE IN COARSE AGGREGATE BED CONNECTED TO STORM DRAIN. GRAVEL BED MUST EXTEND 6" BELOW AND 12" TO THE SIDE AND TOP OF THE PIPE.
4. IF NO UNDERDRAIN, LOW FLOW DRAIN SHALL EXTEND ENTIRE LENGTH OF SWALE AND SHALL HAVE A DEPTH OF 6" MINIMUM AND WIDTH NO MORE THAN 5% SWALE BOTTOM WIDTH. ANCHORED PLATE FLOW SPREADER IF USED, SHALL HAVE V-NOTCHES (MAX TOP WIDTH = 5% OF SWALE WIDTH) OR HOLES TO ALLOW PREFERENTIAL EXIT OF LOW FLOWS.
5. INSTALL CHECK DAMS OR GRADE CONTROL STRUCTURES FOR SLOPES > 2% AT 50' MAXIMUM SPACING TO ACHIEVE A MAXIMUM EFFECTIVE LONGITUDINAL SLOPE OF 2%. FLOW SPREADERS SHALL BE PROVIDED AT INLET AND AT THE BASE OF EACH CHECK DAM.
6. INSTALL ENERGY DISSIPATOR AT THE INLET OF VEGETATED SWALE.
7. SWALE LENGTH SHALL LENGTH REQUIRED TO PROVIDE 7 MINUTES RESIDENCE TIME.
8. INSTALL APPROPRIATE OUTLET STRUCTURE. ACCOMODATE LOW FLOW CHANNEL AND/OR UNDERDRAIN (IF PRESENT).
9. AMEND SOILS WITH 2" OF COMPOST TILLED INTO 6" OF NATIVE SOIL UNLESS NATIVE SOIL ORGANIC CONTENT > 10%.

Geosyntec consultants

Figure 6-12: Vegetated Swale

Limitations

1) Compatibility with flood control - swales should not interfere with flood control functions of existing conveyance and detention structures.

2) Vegetation - select vegetation appropriately based on irrigation requirements and exposure (shady versus sunny areas). A thick vegetative cover is needed for vegetated swales to function properly. Native and drought tolerant plants are recommended.

3) Drainage area - each vegetated swale can treat a relatively small drainage area. Large areas should be divided and treated using multiple swales.

Design Criteria

Vegetated swales should be designed according to the requirements listed in Table 6-20 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-20: Vegetated Swale Filter Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design flow rate (SQDF)	cfs	See Section 2 and Appendix E for calculating SQDF.
Swale Geometry	-	Trapezoidal
Minimum bottom width	feet	2
Maximum bottom width	feet	10; if greater than 10 must use swale dividers; with dividers, max is 16
Minimum length	feet	sufficient length to provide minimum contact time
Minimum slope in flow direction	%	0.2 (provide underdrains for slopes less < 0.5%)
Maximum slope in flow direction	%	2.0 (provide grade-control checks for slopes > 2.0)
Maximum flow velocity	ft/sec	1.0 (water quality treatment); 3.0 (flood conveyance)
Maximum depth of flow for water quality treatment	inches	3 to 5 (1 inch below top of grass)
Minimum residence (contact) time	minutes	7 (provide sufficient length to yield minimum residence time)
Vegetation type	--	Varies (see vegetation section below); Native and drought tolerant plants are recommended
Vegetation height	inches	4 to 6 (trim or mow to maintain height)

Sizing Criteria

The flow capacity of a vegetated swale is a function of the longitudinal slope (parallel to flow), the resistance to flow (i.e. Manning's roughness), and the cross sectional area. The cross section is normally approximately trapezoidal and the area is a function of the bottom width and side slopes. The flow capacity of vegetated swales should be such that the SQDF will not exceed a flow depth of 2/3 the height of the vegetation within the swale or 4 inches at the SQDF. Once design criteria have been selected, the resulting flow depth for the SQDF is checked. If the depth restriction is exceeded, swale parameters (e.g. longitudinal slope, width) are adjusted to reduce the flow depth.

Procedures for sizing vegetated swales are summarized below. A vegetated swale sizing worksheet and example are also provided.

Step 1: Select design flows

The swale sizing is based on the SQDF (see Section 2 and Appendix E).

Step 2: Calculate swale bottom width

The swale bottom width (b) is calculated based on Manning's equation for open-channel flow. This equation can be used to calculate discharges (Q) as follows:

$$Q = \frac{1.49AR^{0.67}S^{0.5}}{n} \qquad \text{(Equation 6-27)}$$

Where:

Q	=	flow rate (cfs)
n	=	Manning's roughness coefficient (unitless)
A	=	cross-sectional area of flow (ft2)
R	=	hydraulic radius (ft) = area divided by wetted perimeter
S	=	longitudinal slope (ft/ft)

For shallow flow depths in swales, channel side slopes are ignored in the calculation of bottom width. Use the following equation (a simplified form of Manning's formula) to estimate the swale bottom width (b):

$$b = \frac{SQDF * n_{wq}}{1.49y^{0.67}s^{0.5}} \qquad \text{(Equation 6-28)}$$

Where:

b	=	bottom width of swale (ft)

SQDF	=	stormwater quality design flow (cfs)
n_{wq}	=	Manning's roughness coefficient for shallow flow conditions = 0.2 (unitless)
y	=	design flow depth (ft)
s	=	longitudinal slope (along direction of flow) (ft/ft)

Proceed to Step 3 if the bottom width is calculated to be between 2 and 10 feet. A minimum 2-foot bottom width is required. Therefore, if the calculated bottom width is less than 2 feet, increase the width to 2 feet and recalculate the design flow depth y using the Equation 6-18, where *SQDF*, n_{wq}, and s are the same values as used above, but b = 2 feet.

The maximum allowable bottom width is 10 feet. Therefore, if the calculated bottom width exceeds 10 feet, then one of the following steps is necessary to reduce the design bottom width:

1) Increase the longitudinal slope (s) to a maximum of 2 feet in 100 feet (0.02 feet per foot).
2) Increase the design flow depth (y) to a maximum of 4 inches.
3) Place a divider lengthwise along the swale bottom (Figure 6-11) at least three-quarters of the swale length (beginning at the inlet), without compromising the design flow depth and swale lateral slope requirements. The swale width can be increased to an absolute maximum of 16 feet if a divider is provided.

Step 3: Determine design flow velocity

To calculate the design flow velocity (V_{wq}) through the swale, use the flow continuity equation:

$$V_{wq} = SQDF/A_{wq} \qquad \text{(Equation 6-29)}$$

Where:

V_{wq}	=	design flow velocity (fps)
SQDF	=	stormwater quality design flow (cfs)
A_{wq}	=	$by + Zy^2$ = cross-sectional area (ft^2) of flow at design depth, where Z = side slope length per unit height (e.g., Z = 3 if side slopes are 3H:1V)

If the design flow velocity exceeds 1 foot per second, go back to Step 2 and modify one or more of the design parameters (longitudinal slope, bottom width, or flow depth) to reduce the design flow velocity to 1 foot per second or less. If the design flow velocity is calculated to be less than 1 foot per second, proceed to Step 4. *Note: It is desirable to have the design velocity as low as possible, both to improve treatment effectiveness and to reduce swale length requirements.*

Step 4: Calculate swale length

Use the following equation to determine the necessary swale length (L) to achieve a hydraulic residence time of at least 7 minutes:

$$L = 60 t_{hr} V_{wq} \quad \text{(Equation 6-30)}$$

Where:

L	=	minimum allowable swale length (ft)
t_{hr}	=	hydraulic residence time (min)
V_{wq}	=	design flow velocity (fps), calculated by Equation 6-19

If there is adequate space on the site to accommodate a larger swale, consider using a greater length to increase the hydraulic residence time and improve the swale's pollutant removal capability. If the calculated length is too long for the site, or if it would cause layout problems, such as encroachment into shaded areas, proceed to Step 5 to further modify the layout. If the swale length can be accommodated on the site (meandering may help), proceed to Step 6.

Step 5: Adjust swale layout to fit on site

If the swale length calculated in Step 4 is too long for the site, the length can be reduced (to a minimum of 100 feet) by increasing the bottom width up to a maximum of 16 feet, as long as the 10 minute retention time is retained. However, the length cannot be increased in order to reduce the bottom width because Manning's depth-velocity-flow rate relationships would not be preserved. If the bottom width is increased to greater than 10 feet, a low flow dividing berm is needed to split the swale cross section in half to prevent channelization.

Length can be adjusted by calculating the top area of the swale and providing an equivalent top area with the adjusted dimensions.

1) Calculate the swale treatment top area (A_{top}), based on the swale length calculated in Step 4:

$$A_{top} = (b_i + b_{slope}) L_i \quad \text{(Equation 6-31)}$$

Where:

A_{top}	=	top area (ft^2) at the design treatment depth
b_i	=	bottom width (ft), calculated in Step 2 using Equation 6-18
b_{slope}	=	the additional top width (ft) above the side slope for the design water depth (for 3:1 side slopes and a 4-inch water depth, b_{slope} = 2 feet)
L_i	=	initial length (ft) calculated in Step 4 using Equation 6-30

2) Use the swale top area and a reduced swale length (L_f) to increase the bottom width, using the following equation:

$$L_f = A_{top} / (b_f + b_{slope}) \qquad \text{(Equation 6-32)}$$

Where:

L_f	=	reduced swale length (ft)
b_f	=	increased bottom width (ft)

3) Recalculate V_{wq} according to Step 3 using the revised cross-sectional area A_{wq} based on the increased bottom width (b_f). Revise the design as necessary if the design flow velocity exceeds 1 foot per second.

4) Recalculate to ensure that the 10 minute retention time is retained.

Step 6: Provide conveyance capacity for flows higher than SQDF

Vegetated swales may be designed as flow-through channels that convey flows higher than the SQDF, or they may be designed to incorporate a high-flow bypass upstream of the swale inlet. A high-flow bypass usually results in a smaller swale size. If a high-flow bypass is provided, this step is not needed. If no high-flow bypass is provided, proceed with the procedure below. A flow splitter structure design is described in Appendix F.

1) Check the swale size to determine whether the swale can convey the flood control design storm peak flow (Refer to Ventura County Hydrology Manual, revised 2006).

2) The peak flow velocity of the flood control design storm (see Ventura County Hydrology Manual revised 2006) should be less than 3.0 feet per second. If this velocity exceeds 3.0 feet per second, return to Step 2 and increase the bottom width or flatten the longitudinal slope as necessary to reduce the flood control design storm peak flow velocity to 3.0 feet per second or less. If the longitudinal slope is flattened, the swale bottom width must be recalculated (Step 2) and must meet all design criteria.

Geometry and Size

1) In general, a trapezoidal channel shape should be assumed for sizing calculations above, but a more naturalistic channel cross-section is preferred.

2) Swales designed for water quality treatment purposes only are usually fairly shallow, generally less than 1 ft. Therefore, a side slope of 2:1 (H:V) can be used and is acceptable.

3) Swales shall be greater than 100 feet in length. The vegetated swale can be shorter than 100 feet if it is used for pretreatment only (i.e., prior to infiltration). Length can be increased by meandering the swale.

4) The minimum swale bottom width shall be 2 feet to allow for ease of mowing.

5) The maximum swale bottom width shall be limited to 10 feet, unless a swale divider is provided, then the maximum bottom width can be a maximum of 16 feet wide. The swale width is calculated without the swale diving berm. *The intention is that experience shows that when the width exceeds about 10 feet, it is difficult to keep the water from concentrating in low flow channels. It is also difficult to construct the bottom level without sloping to one side. Vegetated swales are best constructed by leveling the bottom after excavating. A single-width pass with a front-end loader produces a better result than a multiple-width pass.*

6) Swales that are required to convey flood flow as well as the SQDF should be sized to convey the flood control design storm and include a provision of freeboard as required by the local approval authority.

7) Gradual meandering bends in the swale are desirable for aesthetic purposes and to promote slower flow.

Bottom Slope

1) The longitudinal slope (along the direction of flow) should be between 1% and 6%.

2) If longitudinal slopes are less than 1.5% and the soils are poorly drained (e.g., silts and clays), then underdrains should be provided. A soils report to verify soils properties should be provided for swales less than 1.5%.

3) If longitudinal slope exceeds 2%, check dams with vertical drops of 12 inches or less should be provided to achieve a bottom slope of 2% or less between the drop structures.

4) The lateral (horizontal) slope at the bottom of the swale should be zero (flat) to discourage channeling.

Water Depth and Dry Weather Flow Drain

1) Water depth should not exceed 4 inches (or 2/3 of the expected vegetation height), except for frequently mowed turf swales, in which the depth should not exceed 2 inches.

2) The swale length must provide a minimum hydraulic residence time of 7 minutes.

3) A low flow drain should be provided if the potential for dry weather flows exists. The low flow drain should extend the entire length of the swale. The drain should have a minimum depth of 6 inches, and a width no more than 5% of the calculated swale bottom width. The width of the drain should be in addition to the required bottom width. The flow spreader at the swale inlet should have v-notches (maximum top width = 5% of swale width) or holes to allow preferential exit of low flows into the drain, if applicable. If an underdrain or gravel drainage layer is installed as discussed below, the low flow drain should be omitted.

Swale Inflow and Design Capacity

1) Whenever possible, inflow should be directed towards the upstream end of the swale and should, at a minimum, occur evenly over the length of the swale. Swale inflow design should provide for positive drainage into the swale to function on the long-term with minimal maintenance.

2) On-line vegetated swales should be designed to convey flow rates up to the post-development peak stormwater runoff discharge rate (flow rate) for the 100-yr 24-hour storm event, with appropriate freeboard (see Ventura County Hydrology Manual, revised 2006).

3) Off-line vegetated swales should be designed to convey the flow-based SQDF by using a flow diversion structure (e.g., flow splitter) which diverts the SQDF to the off-line vegetated swale designed to handle SQDF. Freeboard for off-line swales is not required, but should be provided if space is available. Flow splitter design specifications are described in Appendix F.

Energy Dissipation

1) Vegetated swales may be designed either on-line or off-line. If the facility is on-line, velocities should be maintained below the maximum design flow velocity of 3 feet per second to prevent scour and resuspension of deposited sediments.

2) The maximum flow velocity under the stormwater quality design flow rate should not exceed 1.0 foot per second. *The intention is that this maximum SQDV promotes settling and keeps vegetation upright.*

3) This velocity limitation combined with a maximum depth of 4 inches and bottom width of 10 feet results in a recommended maximum flow capacity of about 3.3 cfs,

after accounting for the side slopes. The contributory drainage area to each swale is limited so as not to exceed this recommended maximum flow capacity.

4) The maximum flow velocity during the 100-yr 24-hr storm event should not exceed 3.0 foot per second. This can be accomplished by:

   a. Splitting roadside swales near high points in the road so that flows drain in opposite directions, mimicking flow patterns on the road surface.

   b. Limiting tributary areas to long swales by diverting flows throughout the length of the swale at regular intervals, to the downstream stormwater conveyance system.

5) A flow spreader (see "Flow Spreaders" below) should be used at the inlet so that the entrance velocity is quickly dissipated and the flow is uniformly distributed across the whole swale. Energy dissipation controls should be constructed of sound materials such as stones, concrete, or proprietary devices that are rated to withstand the energy of the influent flows.

6) If check dams are used to reduce the longitudinal slope, a flow spreader should be provided at the toe of each vertical drop, with specifications described below.

7) If flow is to be introduced through curb cuts, place pavement approximately one inch above the elevation of the vegetated areas. Curb cuts should be at least 12 inches wide to prevent clogging.

Flow Spreaders

1) An anchored plate flow spreader or similar device should be provided at the inlet to the swale. Equivalent methods for spreading flows evenly throughout the width of the swale are acceptable.

2) The top surface of the flow spreader plate should be level, projecting a minimum of 2 inches above the ground surface of the water quality facility, or v-notched with notches 6 to 10 inches on center and 1 to 4 inches deep (use shallower notches with closer spacing).

3) A flow spreader plate should extend horizontally beyond the bottom width of the facility to prevent water from eroding the side slope. The plate should have a row of horizontal perforations at its base to prevent ponding for long durations. The horizontal extent should be such that the bank is protected for all flows up to the 100-yr 24-hr storm event (on-line swales) or the maximum flow that will enter the water quality facility (off-line swales).

4) Flow spreader plates should be securely fixed in place.

5) Flow spreader plates may be made of either concrete, stainless steel, or other durable material.

6) Anchor posts should be 4-inch square concrete, tubular stainless steel, or other material resistant to decay.

Check Dams

If check dams are required, they can be designed using a number of different materials, including riprap, earthen berms, or removal stop logs. Where vegetated swales parallel urban streets, the check dam can double as a crossing walk so that pedestrians have a pathway from the parked car to the building.

Check dams must be placed as to achieve the desired slope (1 to 6%) at a maximum of 50 feet apart. Check dams should be no higher than 12 inches. If riprap is used, the material should consist of well-graded stone consisting of a mixture of rock sizes. The following is an example of an acceptable gradation:

Particle Size	% Passing
24 inch	100
15 inch	75
9 inch	50
4 inch	10

Underdrains

If underdrains (not to be confused with a dry weather flow drain) are required, then they should meet the following criteria:

1) Underdrains should be made of slotted, polyvinyl chloride (PVC) pipe (PVC SDR 35 or approved equivalent). *The intention is that in comparison to round-hole perforated pipe, slotted underdrains provide greater intake capacity, clog resistant drainage, and reduced entrance velocity into the pipe, thereby reducing the chances of solids migration.*

2) Slotted pipe should have 2 to 4 rows of slots cut perpendicular to the axis of the pipe or at right angles to the pitch of corrugations. Slots should be 0.04 to 0.1 inch and should have a length of 1 to 1.25 inches. Slots should be longitudinally spaced such that the pipe has a minimum of one square inch of opening per linear foot of pipe.

3) Underdrains should be sloped at a minimum of 0.5%.

4) The underdrain pipe should be 6 inches or greater in diameter, so it can be cleaned without damage to the pipe. Clean-out risers with diameters equal to the underdrain pipe should be placed at the terminal ends of the underdrain and can be incorporated into the flow spreader and outlet structure to minimize maintenance obstacles in the swale. Intermediate clean-out risers may also be placed in the check dams or grade control structures. The cleanout risers should be capped with a lockable screw cap.

5) The underdrain should be placed parallel to the swale bottom and backfilled and underbedded with six inches of drain rock. The following coarse aggregate should be used to provide a gravel blanket and bedding for the underdrain pipe to provide a 1 foot minimum depth around the top and sides of the slotted pipe.

Sieve size	Percent Passing
¾ inch	100
¼ inch	30-60
US No. 8	20-50
US No. 50	3-12
US No. 200	0-1

6) At the option of the designer/geotechnical engineer, the drain rock may be wrapped in a geotextile fabric meeting the following minimum materials requirements. If a geotextile fabric is used, it should pass 75 gal/min/ft^2, should not impede the infiltration rate of the soil medium, and should meet the following minimum materials requirements.

Geotextile Property	Value	Test Method
Trapezoidal Tear (lbs)	40 (min)	ASTM D4533
Permeability (cm/sec)	0.2 (min)	ASTM D4491
AOS (sieve size)	#60 - #70 (min)	ASTM D4751
Ultraviolet resistance	70% or greater	ASTM D4355

Preferably, aggregate should be used in place of geotextile fabric to reduce the potential for clogging. This aggregate layer should consist of 2 to 4 inches of washed sand underlain with 2 inches of choking stone (Typically #8 or #89 washed).

7) The underdrain should drain freely to an acceptable discharge point. The underdrain can be connected to a downstream open conveyance (vegetated swale), to another bioretention cell as part of a connected treatment system, daylight to a vegetated dispersion area using an effective flow dispersion device, stored for rainwater harvesting, or to a storm drain.

Gravel Drainage Layer

To increase volume reduction and if soil conditions allow (infiltration rate > 0.5 in/hr), omit the low flow drain or underdrain and install an appropriately sized gravel drainage layer (typically a washed 57 stone) beneath the swale to achieve desired volume reduction goals. Where slopes are greater than 1%, the gravel drainage layer should be installed in combination with check dams (e.g., drop structures) to slow the flow in the swale and allow for infiltration into the gravel drainage layer and then into the subsurface. The base of the drainage layer should have zero slope. The drawdown time in the gravel drainage layer should not exceed 72 hours. The soil and gravel layers should be separated with a geotextile filter fabric or a thin, 2 to 4 inch layer of pure sand and a thin layer (nominally two inches) of choking stone (such as #8). Sizing of the gravel drainage layer is based on volume reduction requirements.

Swale Divider

1) If a swale divider is used, the divider should be constructed of a firm material that will resist weathering and not erode, such as concrete, plastic, or compacted soil seeded with grass. Treated timber should not be used. Selection of divider material should take into account maintenance activities, such as mowing.

2) The divider should have a minimum height of 1 inch greater than the stormwater quality design water depth.

3) Earthen berms should be no steeper than 2H:1V.

4) Material other than earth should be embedded to a depth sufficient to be stable.

Soils

Swale soils should be amended with 2 inches of compost, unless the organic content is already greater than 10%. The compost should be mixed into the native soils to a depth of 6 inches to prevent soil layering and washout of compost. The compost will contain no sawdust, green or under-composted material, or any other toxic or harmful substance. It should contain no un-sterilized manure, which can lead to high levels of pathogen indictors (coliform bacteria) in the runoff.

Vegetation

Swales must be vegetated in order to provide adequate treatment of runoff via filtration. Vegetation, when chosen and maintained appropriately, also improves the aesthetics of a site. It is important to maximize water contact with vegetation and the soil surface.

1) The swale area should be appropriately vegetated with a mix of erosion-resistant plant species that effectively bind the soil. A diverse selection of low growing plants that thrive under the specific site, climatic, and watering conditions should be specified. A mixture of dry-area and wet-area grass species that can continue to grow through silt deposits is most effective. Native or adapted grasses are preferred because they generally require less fertilizer, limited maintenance, and are more drought-resistant than exotic plants. When appropriate, swales that are integrated within a project may use turf or other more intensive landscaping, while swales that are located on the project perimeter, within a park, or close to an open space area are encouraged to be planted with a more naturalistic plant palette.

2) Trees or shrubs may be used in the landscape as long as they do not over-shade the turf.

3) Above the design treatment elevation, a typical lawn mix or landscape plants can be used provided they do not shade the swale vegetation.

4) Irrigation is required if the seed is planted in the spring or summer. Use of a permanent irrigation system may help provide maximal water quality performance. Drought-tolerant grasses should be specified to minimize irrigation requirements.

5) Vegetative cover should be at least 4 inches in height, ideally 6 inches. Swale water depth should ideally be 2/3 of the height of the shortest plant species.

6) Locate the swale in an area without excessive shade to avoid poor vegetative growth. For moderately shaded areas, shade tolerant plants should be used.

7) Locate the swale away from large trees that may drop excessive leaves or needles, which may smother the grass or impede the flow through the swale. Landscape planter beds should be designed and located so that soil does not erode from the beds and enter a nearby swale.

Maintenance Access

1) Access to the swale inlet and outlet should be safely provided, with ample room for maintenance and operational activities.

Operations and Maintenance

1) Inspect vegetated swales for erosion or damage to vegetation after every storm greater than 0.75 inches for on-line swales and at least twice annually for off-line swales, preferably at the end of the wet season to schedule summer maintenance and in the fall to ensure readiness for winter. Additional inspection after periods of heavy runoff is recommended. Each swale should be checked for debris and litter and areas of sediment accumulation (see Appendix I for a vegetated swale inspection and maintenance checklist).

2) Swale inlets (curb cuts or pipes) should maintain a calm flow of water entering the swale. Remove sediment as needed at the inlet, if vegetation growth is inhibited in greater than 10% of the swale or if the sediment is blocking even distribution and entry of the water. Following sediment removal activities, replanting and/or reseeding of vegetation may be required for reestablishment.

3) Flow spreaders should provide even dispersion of flows across the swale. Sediments and debris should be removed from the flow spreader if blocking flows. Splash pads should be repaired if needed to prevent erosion. Spreader level should be checked and releveled if necessary.

4) Side slopes should be maintained to prevent erosion that introduces sediment into the swale. Slopes should be stabilized and planted using appropriate erosion control measures when native soil is exposed or erosion channels are formed.

5) Swales should drain within 48 hours of the end of a storm. Till the swale if compaction or clogging occurs and revegetate. If a perforated underdrain pipe is present, it should be cleaned if necessary.

6) Vegetation should be healthy and dense enough to provide filtering, while protecting underlying soils from erosion:

   - Mulch should be replenished as needed to ensure survival of vegetation.

   - Vegetation, large shrubs or trees that interfere with landscape swale operation should be pruned.

   - Fallen leaves and debris from deciduous plant foliage should be removed.

   - Grassy swales should be mowed to 4 to 6 inches height. Grass clippings should be removed.

   - Invasive vegetation, such as Alligatorweed (*Alternanthera philoxeroides*), Halogeton (*Halogeton glomeratus*), Spotted Knapweed (*Centaurea maculosa*), Giant Reed (*Arundo donax*), Castor Bean (*Ricinus communis*), Perennial Pepperweed (*Lepidium latifolium*), and Yellow Starthistle (*Centaurea solstitalis*) should be removed and replaced with non-invasive species. Invasive species should never contribute more than 10% of the vegetated area. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website or the California Invasive Plant Council website at www.cal-ipc.org.

   - Dead vegetation should be removed if greater than 10% of area coverage or when swale function is impaired. Vegetation should be replaced and established before the wet season to maintain cover density and control erosion where soils are exposed.

7) Check dams (if present) should control and distribute flow across the swale. Causes for altered water flow and/or channelization should be identified and obstructions cleared. Check dams and swale should be repaired if damaged.

8) The vegetated swale should be well maintained. Trash and debris, sediment, visual contamination (e.g., oils), noxious or nuisance weeds, should all be removed.

BIO-4: Vegetated Filter Strip

Filter strips are vegetated areas designed to treat sheet flow runoff from adjacent impervious surfaces or intensive landscaped areas such as golf courses. Filter strips decrease runoff velocity, filter out total suspended solids and associated pollutants, and provide some infiltration into underlying soils. While some assimilation of dissolved constituents may occur, filter strips are generally more effective in trapping sediment and particulate-bound metals, nutrients, and pesticides. Filter strips are more effective when the runoff passes through the vegetation and thatch layer in the form of shallow, uniform flow. Biological and chemical processes may help break down pesticides, uptake metals, and use nutrients that are trapped in the filter.

Vegetated filter strip captures runoff from freeway

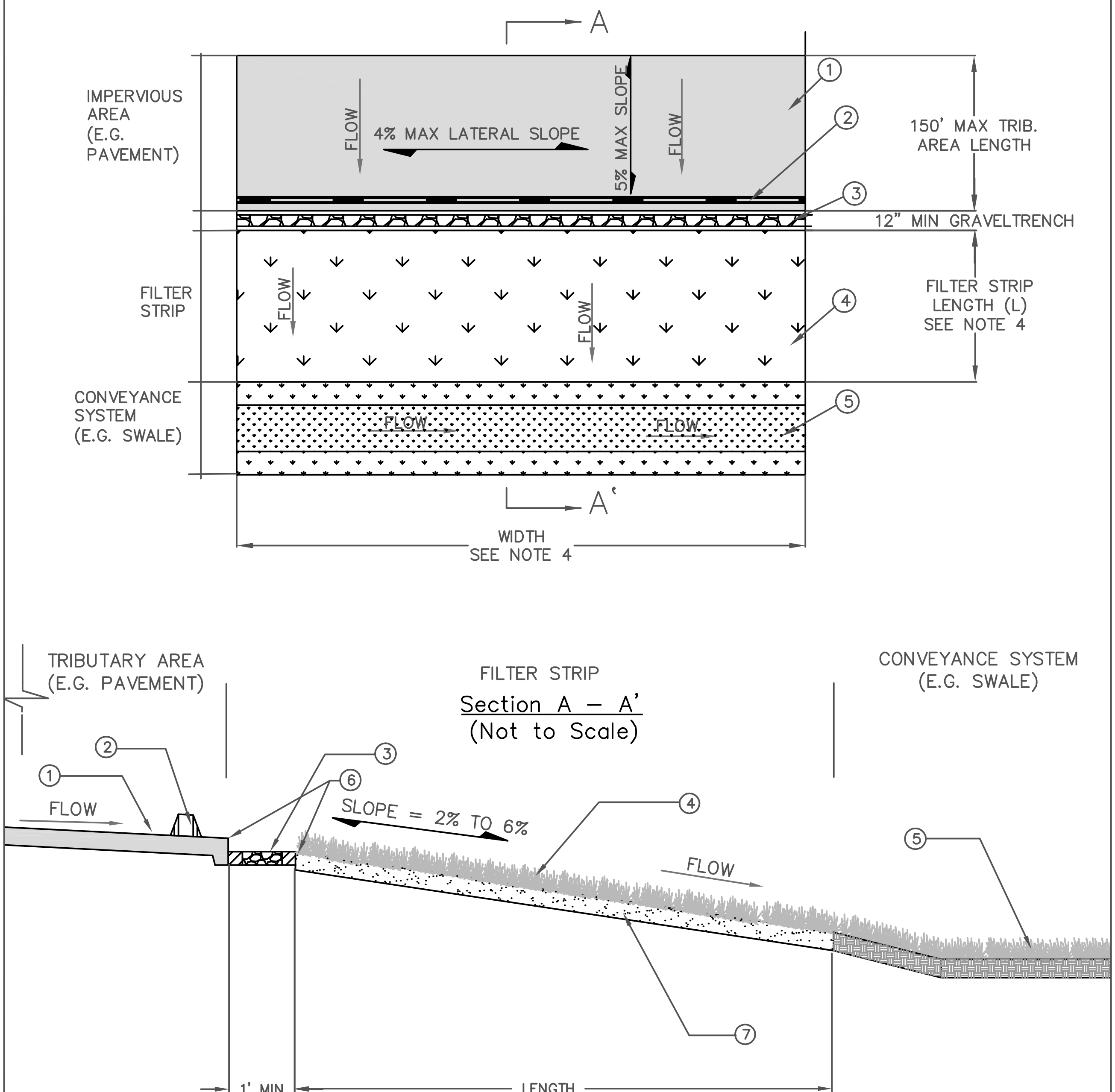
Photo Credit: Washington Department of Transportation

Applications

- Areas adjacent to parking lots and driveways
- Road medians and shoulders

Preventative Maintenance

- Remove excess sediment
- Stabilize/repair minor erosion and scouring
- Remove trash and debris
- Mow regularly



NOTES:

1. MAXIMUM LENGTH OF IMPERVIOUS TRIBUTARY AREA (PER UNIT WIDTH OF FILTER STRIP) SHALL BE 150'.
2. OPTIONAL NOTCHED CURB SPREADERS MAY BE USED IN CONJUNCTION WITH GRAVEL FLOW SPREADER.
3. GRAVEL FLOW SPREADER 6" DEEP BY 12" WIDE MIN SHALL BE PROVIDED.
4. VEGETATED FILTER STRIP SURFACE. SLOPE SHALL BE BETWEEN 2–4%. LENGTH OF STRIP MUST BE BETWEEN 15' AND 150'. WIDTH MUST BE EQUAL OR GREATER THAN THE WIDTH OF THE TRIBUTARY AREA.
5. INSTALL SWALE OR OTHER CONVEYANCE SYSTEM DOWNSTREAM OF FILTER STRIP.
6. TOP OF FILTER STRIP SHALL BE >1" BELOW TOP OF ADJACENT PAVEMENT.
7. AMEND SOILS WITH 2" OF COMPOST TILLED INTO 6" OF NATIVE SOIL UNLESS NATIVE SOIL ORGANIC CONTENT > 10%.

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Figure 6-13: Vegetated Filter Strip

Limitations

The following describes limitations for vegetated filter strips:

- High flow velocity - steep terrain and/or large tributary area may cause concentrated, erosive flows.
- Sheet flow - shallow, evenly-distributed flow across the entire width of the filter strip is required. Filter strips are designed to treat small areas. The maximum flow path from a contributing impervious surface should not exceed 150 feet. Flows should enter as sheet flow and not exceed a depth of 1 inch.
- Shallow grades – a limited site slope may cause ponding.
- Availability of pervious area adjacent to impervious area - filter strips require sheet flow from impervious areas.

Design Criteria

The main challenge associated with filter strips is maintaining sheet flow, which is critical to the performance of this BMP. If flows are concentrated, then little or no treatment of stormwater runoff is achieved and erosive rilling is likely. The use of a flow spreading device (e.g., gravel trench or level spreader) to deliver shallow, evenly-distributed sheet flow to the strip is required. Vegetated filter strips should be designed according to the requirements listed in Table 6-21 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-21: Vegetated Filter Strip Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design flow (SQDF)	cfs	See Section 2 and Appendix E for calculating SQDF.
Maximum design flow depth	inches	1
Design residence time	minutes	7
Design flow velocity	ft/sec	< 1 ft/sec
Minimum length in flow direction	feet	15 (25 preferred); If sized for pretreatment only, filter strip can be a minimum of 4.
Maximum length (parallel to flow) of tributary area per unit width (perpendicular to flow) of filter strip	feet	150
Minimum slope in flow direction	%	2

Design Parameter	Unit	Design Criteria
Maximum slope in flow direction	%	4
Maximum lateral slope	%	4
Vegetation	-	Turf grass (irrigated) or approved equal
Minimum grass height	inches	2
Maximum grass height	inches	4 (typical) or as required to prevent shading
Elevation of flow spreader	inches	> 1 inch below the pavement surface

Sizing Criteria

The flow capacity of a vegetated filter strips (filter strips) is a function of the longitudinal slope (parallel to flow), the resistance to flow (e.g., Manning's roughness), and the width and length of the filter strip. The slope should be shallow enough to ensure that the depth of water will not exceed 1 inch over the filter strip. Similarly, the flow velocity should be less than 1 ft/sec. Procedures for sizing filter strips are summarized below. A filter strip sizing example is also provided.

Step 1: Calculate the design flow rate

The design flow is calculated based on the SQDF (see Section 2).

Step 2: Calculate the minimum width

Determine the minimum width (W_{min}), perpendicular to flow, allowable for the filter strip and design for that width or larger.

$$Wmin = (SQDF) / (qa,min) \qquad \text{(Equation 6-33)}$$

Where

W_{min} = minimum width of filter strip (and tributary area)

$SQDF$ = design flow (cfs)

$q_{a,min}$ = minimum linear unit application rate, 0.005 cfs/ft

Step 3: Calculate the design flow depth

The design flow depth (d_f) is calculated based on the width and the slope, parallel to the flow path, using a modified Manning's equation as follows:

$$d_f = 12 \times [SQDF * n_{wq} / 1.49 W_{trib} s^{0.5}]^{0.6} \qquad \text{(Equation 6-34)}$$

Where:

d_f	=	design flow depth (inches)
$SQDF$	=	design flow (cfs)
W	=	width of strip (perpendicular to flow = width of impervious surface contributing area (ft))
s	=	slope (ft/ft) of strip parallel to flow, average over the whole width
n_{wq}	=	Manning's roughness coefficient (0.25-0.30)

If d_f is greater than 1 inch (0.083 ft), then a shallower slope is required, or a filter strip cannot be used.

Step 4: Calculate the design velocity

The design flow velocity (V_{wq}) is based on the design flow, design flow depth, and width of the strip:

$$Vwq = SQDF/ (df\ W) \qquad \text{(Equation 6-35)}$$

Where:

$d_{f,ft}$	=	design flow depth (ft) (d_f/12)
$SQDF$	=	stormwater quality design flow (cfs)
W	=	width of strip (perpendicular to flow = width of impervious surface contributing area (ft))

Step 5: Calculate the desired length of the filter strip

Determine the required length (L) to achieve a desired minimum residence time of 7 minutes using:

$$L = 60t_{hr} * V_{wq} \qquad \text{(Equation 6-36)}$$

Where:

L	=	minimum allowable strip length (ft)
t_{hr}	=	hydraulic residence time (min)
V_{wq}	=	design flow velocity (fps) calculated by Equation 6-35

Geometry and Size

1) The width of the filter strip shall extend across the full width of the tributary area. The upstream boundary of the filter should be located contiguous to the developed tributary area.

2) The length (in direction of flow) should be between 15 and 150 feet. A minimum length of 25 feet is preferred. Filter strips used for pretreatment shall be at least 4 feet long (in direction of flow).

3) Filter strips shall be designed on slopes (parallel to the direction of flow) between 2% and 4%; steeper slopes tend to result in concentrated flow. Slopes less than 2% could pond runoff, and in poorly permeable soils, create a mosquito breeding habitat.

4) The lateral slope of strip (parallel to the edge of the pavement, perpendicular to the direction of flow) should be 4% or less.

5) Grading should be even: a filter strip with uneven grading perpendicular to the flow path will develop flow channels over time.

6) The top of the strip should be installed 2 to 5 inches below the adjacent pavement to allow for vegetation and sediment accumulation at the edge of the strip. A beveled transition is acceptable and may be required per roadside design specifications.

7) Both the top and toe of the slope should be as flat as possible to encourage sheet flow and prevent channeling and erosion. For engineered filter strips, the facility surface should be graded flat prior to placement of vegetation.

Energy Dissipation / Level Spreading

Runoff entering a filter strip must not be concentrated. A flow spreader should be installed at the edge of the pavement to uniformly distribute the flow along the entire width of the filter strip.

1) At a minimum, a gravel flow spreader (gravel-filled trench) should be placed between the impervious area contributing flows and the filter strip, and meet the following requirements:

    a. The gravel flow spreader should be a minimum of 6 inches deep and should be 12 inches wide.

    b. The gravel should be a minimum of 1 inch below the pavement surface. *The intention is that this allows sediment from the paved surface to be accommodated without blocking drainage onto the strip.*

2) The gravel flow spreader should be a minimum of 6 inches deep and should be 12 inches wide.

a. Where the ground surface is not level, the gravel spreader must be installed so that the bottom of the gravel trench and the outlet lip are level.

b. Along roadways, gravel flow spreaders must be placed and designed in accordance with County road design specifications for compacted road shoulders.

3) Curb ports and interrupted curbs may only be used in conjunction with a gravel spreader to better ensure that water sheet flows onto the strip, provided:

a. Curb ports use fabricated openings that allow concrete curbing to be poured or extruded while still providing an opening through the curb to admit water to the filter strip. Interrupted curbs are sections of curb placed to have gaps spaced at regular intervals along the total width of the treatment area. Openings or gaps in the curb should be at regular intervals but at least every 6 feet. The width of each opening should be a minimum of 11 inches.

b. At a minimum, gaps should be every 6 feet to allow distribution of flows into the treatment facility before they become too concentrated. The opening should be a minimum of 11 inches. Approximately 15 percent or more of the curb section length should be in open ports, and as a general rule, no opening should discharge more than 10 percent of the overall flow entering the facility.

4) Energy dissipaters are needed in a filter strips if sudden slope drops occur, such as locations where flows in a filter strip pass over a rockery or retaining wall aligned perpendicular to the direction of flow. Adequate energy dissipation at the base of a drop section can be provided by a riprap pad.

Access

1) Access should be provided at the upper edge of a filter strip to enable maintenance of the inflow spreader throughout the strip width and allow access for mowing equipment.

Water Depth and Velocity

1) The design water depth shall not exceed 1 inch.

2) Runoff flow velocities should not exceed approximately 1 foot per second across the filter strip surface.

Soils

Filter strip soils should be amended with 2 inches of compost, unless the organic content is already greater than 10%. The compost should be mixed into the native soils to a depth of 6 inches to prevent soil layering and washout of compost. The compost will contain no sawdust, green or under-composted material, or any other toxic or harmful substance. It should contain no un-sterilized manure which can lead to high levels of potentially pathogenic bacteria in the runoff.

Vegetation

Filter strips must be uniformly graded and densely vegetated with erosion-resistant grasses that effectively bind the soil. Native or adapted grasses are preferred because they generally require less fertilizer and are more drought-resistant than exotic plants. The following vegetation guidelines should be followed for filter strips:

1) Sod (turf) can be used instead of grass seed, as long as there is complete coverage.

2) Irrigation should be provided to establish the grasses.

3) Grasses or turf should be maintained at a height of 2 to 4 inches. Regular mowing is often required to maintain the turf grass cover.

4) Trees or shrubs should not be used in abundance because they shade the turf and impede sheet flow.

Operations and Maintenance

Filter strips mainly require vegetation management. Therefore little special training is needed for maintenance crews. Typical maintenance activities and frequencies include:

1) Inspect strips at least twice annually for erosion or damage to vegetation, preferably at the end of the wet season to schedule summer maintenance and in the fall to ensure the strip is ready for winter. However, additional inspection after periods of heavy runoff is most desirable. The strip should be checked for debris and litter and areas of sediment accumulation (see Appendix I for a vegetated filter strip inspection and maintenance checklist).

2) Mow as frequently as necessary (at least twice a year) for safety and aesthetics or to suppress weeds and woody vegetation.

3) Trash tends to accumulate in strip areas, particularly along roadways. The need for litter removal should be determined through periodic inspection. Litter should always be removed prior to mowing.

4) Regularly inspect vegetated buffer strips for pools of standing water. Vegetated filter strips can become a nuisance due to mosquito breeding in level spreaders (unless designed to dewater completely in less than 72 hours), in pools of standing water if obstructions develop (e.g. debris accumulation, invasive vegetation), and/or if proper drainage slopes are not implemented and maintained.

5) Activities that lead to ruts or depressions on the surface of the filter strip should be prevented or the integrity of the strip should be restored by leveling and reseeding. Examples are vehicle tracks, utility maintenance, and pedestrian (short-cut) tracks.

6) Vegetation should be healthy and dense enough to provide filtering, while protecting underlying soils from erosion:

   - Mulch should be replenished as needed to ensure survival of vegetation.
   - Vegetation, large shrubs or trees that interfere with landscape swale operation should be pruned.
   - Fallen leaves and debris from deciduous plant foliage should be removed.
   - Filter strips should be mowed to 4 to 6 inches height. Grass clippings should be removed.
   - Invasive vegetation, such as Alligatorweed (*Alternanthera philoxeroides*), Halogeton (*Halogeton glomeratus*), Spotted Knapweed (*Centaurea maculosa*), Giant Reed (*Arundo donax*), Castor Bean (*Ricinus communis*), Perennial Pepperweed (*Lepidium latifolium*), and Yellow Starthistle (*Centaurea solstitalis*) should be removed and replaced with non-invasive species. Invasive species should never contribute more than 10% of the vegetated area. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website or the California Invasive Plant Council website at www.cal-ipc.org.
   - Dead vegetation should be removed if greater than 10% of area coverage or when filter strip function is impaired. Vegetation should be replaced and established before the wet season to maintain cover density and control erosion where soils are exposed.

BIO-5: Proprietary Biotreatment

Proprietary biotreatment devices are manufactured treatment BMPs that incorporate plants, soil, and microbes engineered to provide treatment at higher flow rates or volumes and with smaller footprints than their non-proprietary counterparts. Incoming flows are typically pretreated to remove larger particles/debris, filtered through a planting media (mulch, compost, soil, and plants), collected by an underdrain, and delivered to the stormwater conveyance system.

Proprietary Biotreatment Examples

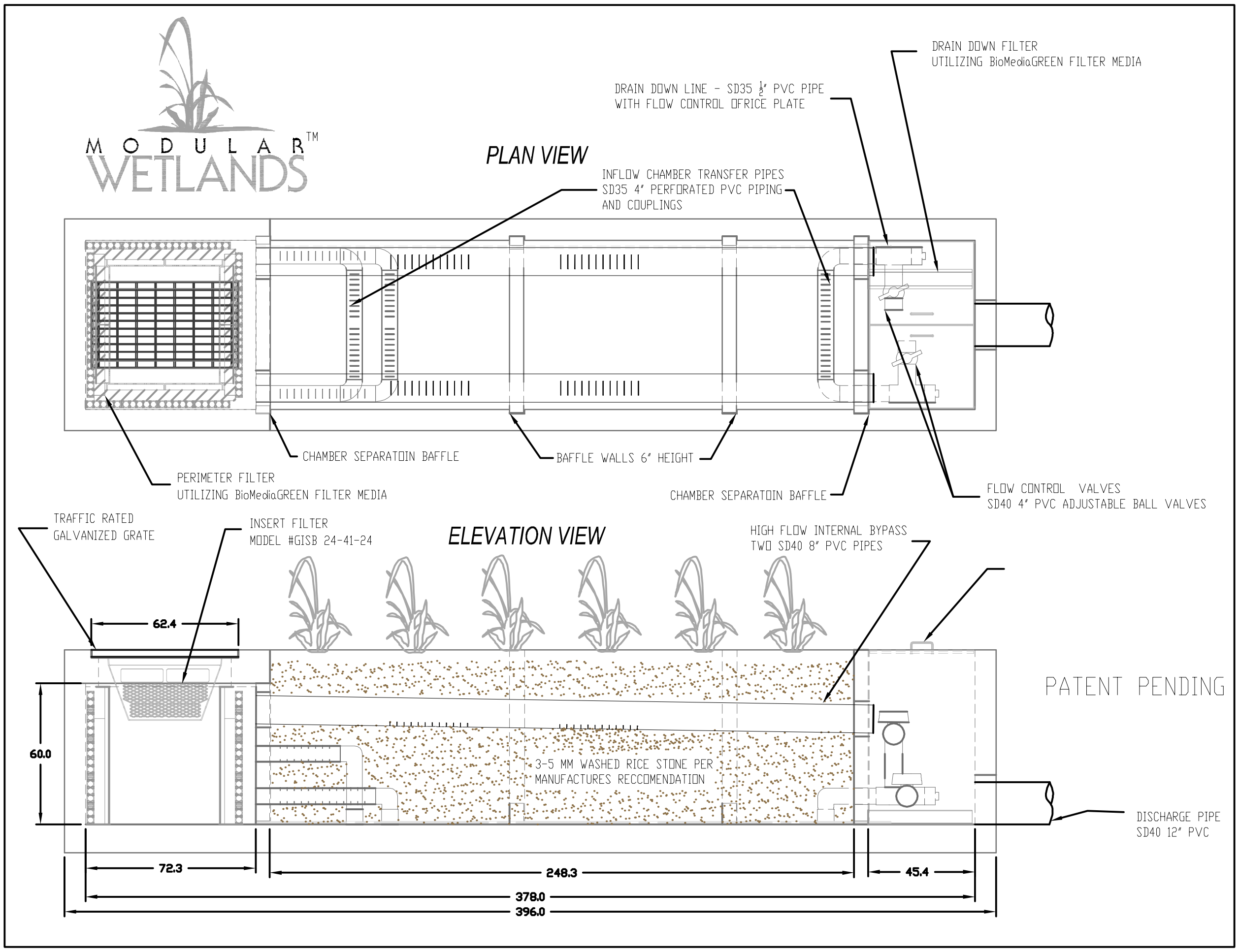
Photo Credits: 1. Filterra®; 2. Stormtreat™

Application

- Parking lot islands
- Pickup/drop off turnarounds
- Roadway curbs

Maintenance

- Filter media replacement
- Sediment, trash, and debris removal
- Mulch replacement
- Vegetation upkeep and replacement



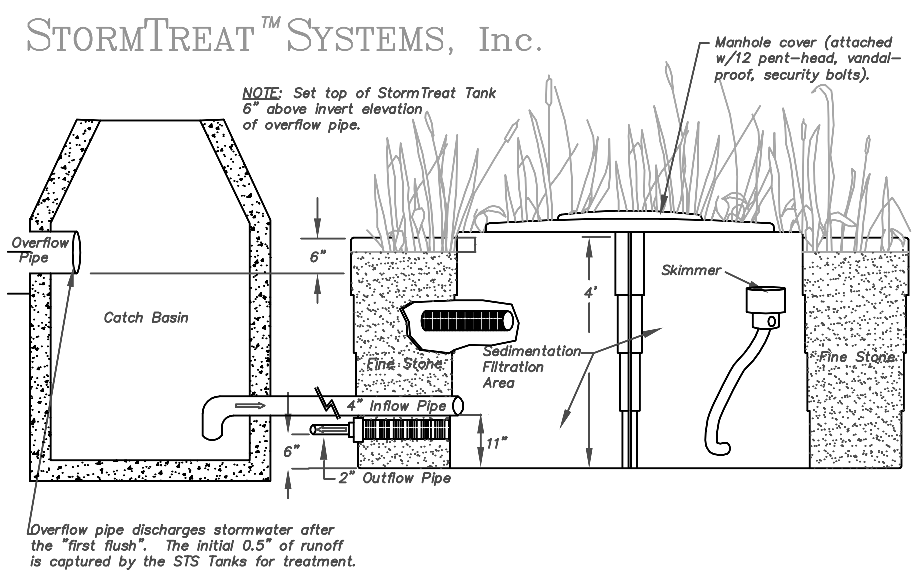


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Figure 6-14: Biotreatment Device

Table 6-22: Proprietary Biotreatment Device Manufacturer Websites

Device	Manufacturer	Website
DeepRoot® Silva Cell	DeepRoot® Urban Landscape Products	www.deeproot.com
Filterra®	Filterra® Bioretention Systems	www.filterra.com
Modular Wetlands (MWS-LINEAR)	Modular Wetlands Systems Inc.	www.modularwetlands.com
StormTreat™	StormTreat Systems Inc.	www.stormtreat.com
UrbanGreen BioFilter	Contech® Construction Products Inc.	www.contech-cpi.com/stormwater/13

Design Criteria

As proprietary biotreatment BMP vendors are constantly updating and expanding their product lines, refer to the specific vendor for the latest design and sizing guidance.

TCM-1: Dry Extended Detention Basin

Dry extended detention (ED) basins are basins whose outlets have been designed to detain the SQDV for 36 to 48 hours to allow sediment particles and associated pollutants to settle and be removed. Dry ED basins do not have a permanent pool. They are designed to drain completely between storm events. They can also be used to provide hydromodification and/or flood control by modifying the outlet control structure and providing additional detention storage. The slopes, bottom, and forebay of dry ED basins are typically vegetated. Without the addition of a sand filter beneath the basin, considerable stormwater volume reduction can still occur, depending on the infiltration capacity of the subsoil.

Extended Detention Basin Application

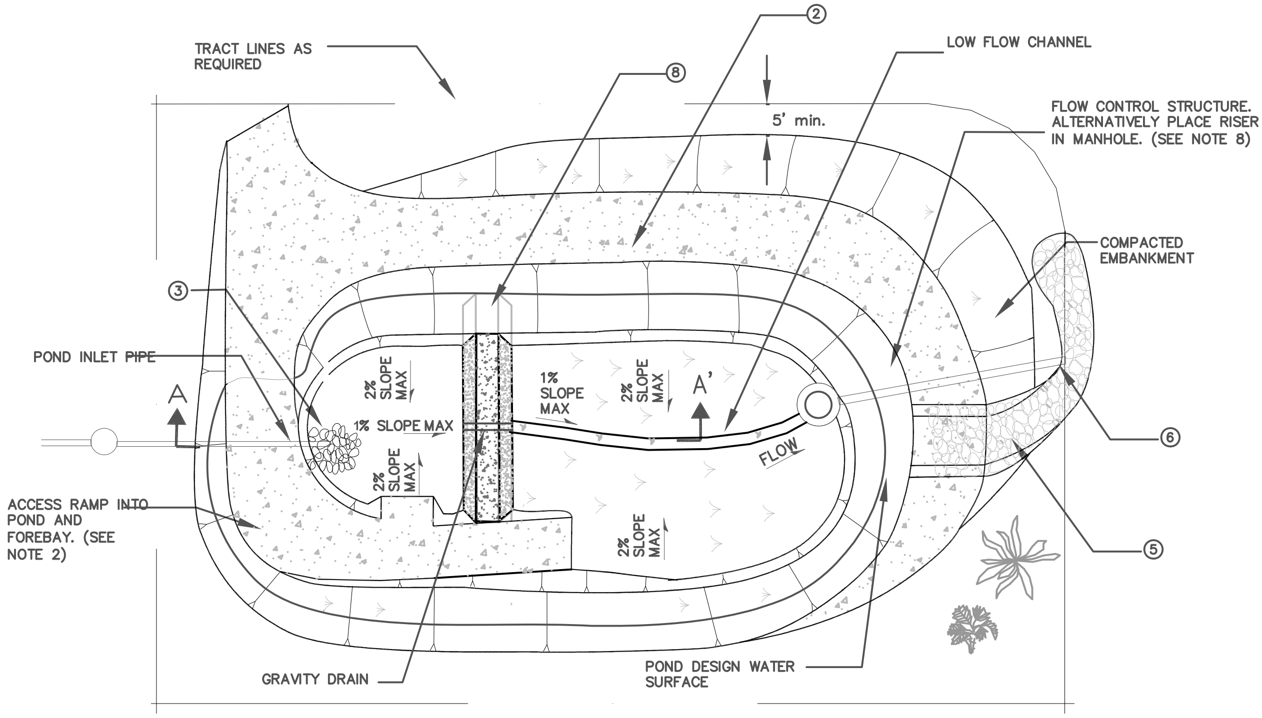
Photo Credit: Geosyntec Consultants

Application

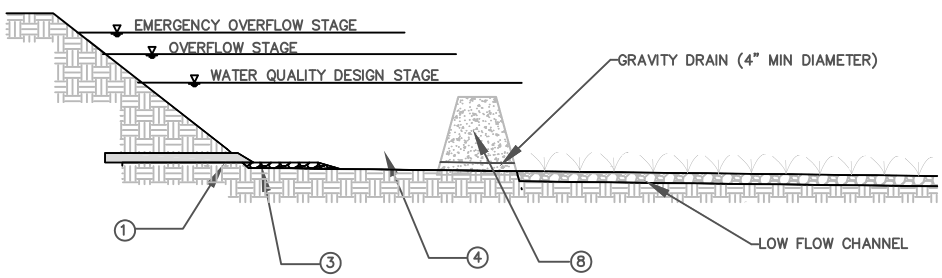
- Adjacent to parking lots
- Road medians and shoulders
- Within open areas or play fields

Preventative Maintenance

- Remove trash and debris, minor sediment accumulation, and obstructions near inlet and outlet structures
- Replace top 2 to 4 inch of sand
- Mow or weed surface of filter



Plan View
(Not to Scale)



Section A – A'
(Not to Scale)

NOTES:

1. INLET PIPE SHALL BE DESIGNED AND LOCATED SO THAT NON–EROSIVE VELOCITIES OCCUR IN THE FOREBAY
2. MAINTENANCE RAMP SHOULD PROVIDE ACCESS TO BOTH THE FOREBAY AND MAIN BASIN.
3. RIP RAP APRON OR OTHER INLET ENERGY DISSIPATION SHALL BE PROVIDED SUCH THAT VELOCITIES IN THE FOREBAY ARE < 4 FT/S.
4. SEDIMENT FOREBAY SHOULD BE SIZED TO PROVIDE 5–15% OF THE TOTAL BASIN VOLUME.
5. EMERGENCY SPILLWAY MUST BE SIZED TO PASS 100–YEAR PEAK FLOW FOR ON–LINE BASINS, AND WATER QUALITY DESIGN FLOW FOR OFF–LINE BASINS.
6. OUTLET PIPE. ENERGY DISSIPATION SHALL BE PROVIDED UNLESS DISCHARGE IS TO PIPE OR HARDENED CHANNEL.
7. OUTLET STRUCTURE SHOULD BE SIZED TO DRAIN WATER QUALITY VOLUME IN 36 – 48 HOURS (SEE FIGURE 2–2 FOR PERFORATED RISER DETAILS). ALTERNATIVELY PLACE RISER STRUCTURE IN A MANHOLE (SEE FIGURE 2–3).
8. INSTALL EARTHEN BERM OR EQUIVALENT. TOP OF BERM SHALL BE 2' MINIMUM BELOW DESIGN WATER QUALITY STAGE. BERM SHALL BE KEYED INTO EMBANKMENT A MINIMUM OF 1' ON BOTH SIDES.

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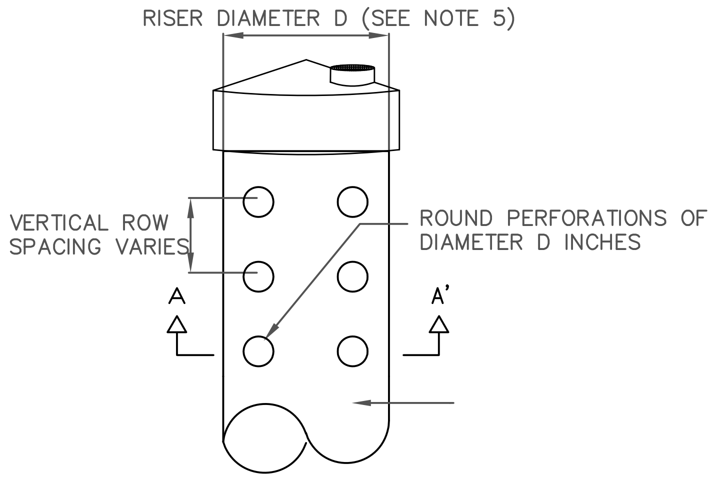
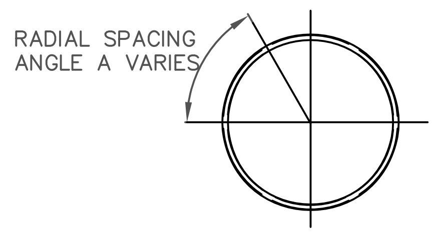
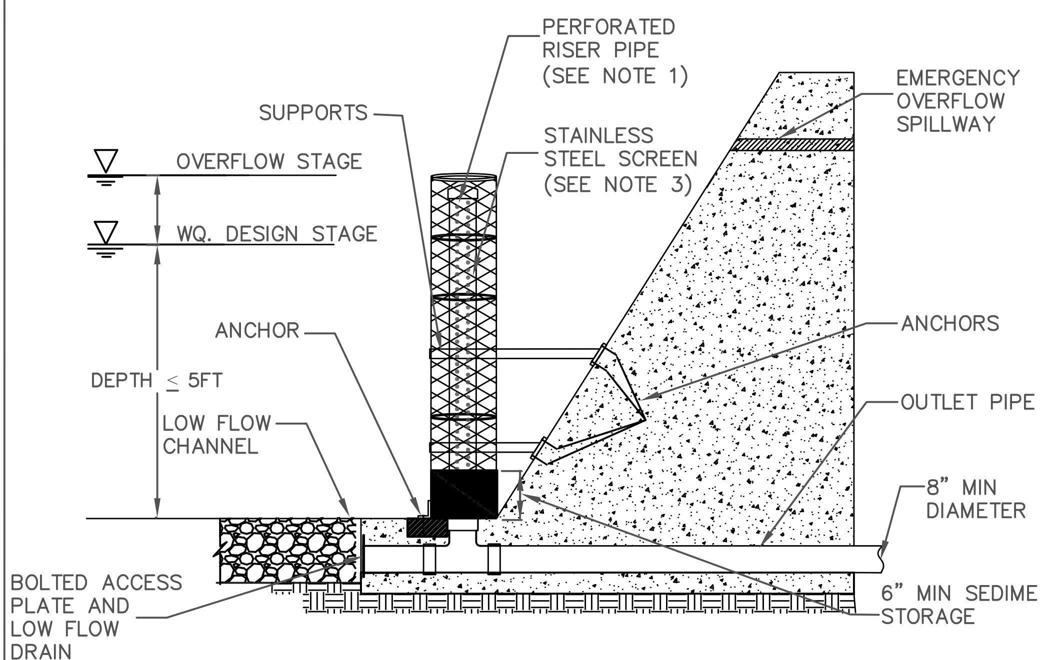
Figure 6-15: Extended Detention Basin



Perforated Riser Detail
(Not to Scale)



Section A – A'
(Not to Scale)



Perforated Riser Outlet Structure
(Not to Scale)

NOTES:

1. RISER PIPE SHALL BE SIZED TO PROVIDE 36 TO 48–HOUR FULL BRIM DRAW DOWN TIME.
2. TOTAL OUTLET CAPACITY: 100–YEAR PEAK FLOW FOR ON–LINE BASINS AND WATER QUALITY DESIGN FLOW FOR OFF–LINE BASINS.
3. SCREEN OPENINGS SHALL BE AT LEAST ¼" AND SHALL NOT EXCEED THE DIAMETER OF THE PERFORATIONS ON THE RISER.
4. RISER PIPE PERFORATION DIAMETER SHALL BE NO LESS THAN ½" AND NO MORE THAN 2"
5. MINIMUM PIPE DIAMETER (D) IS 2'
6. RISER PIPE MATERIAL IS CMP

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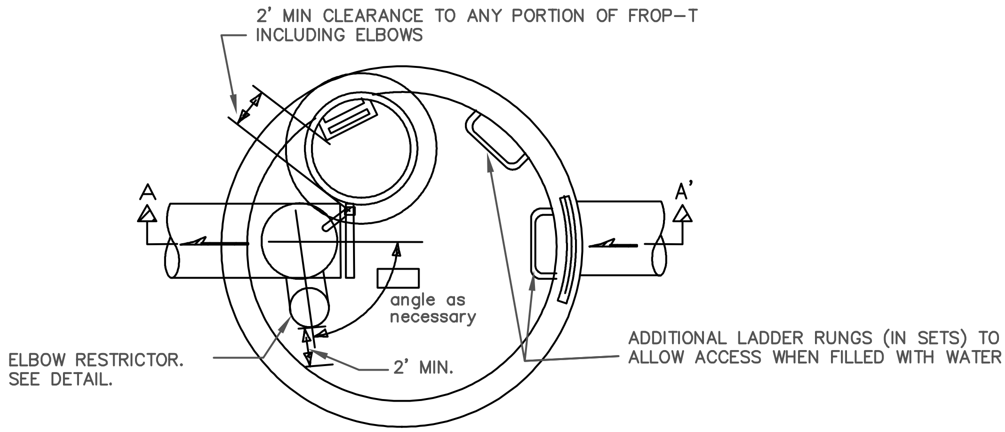
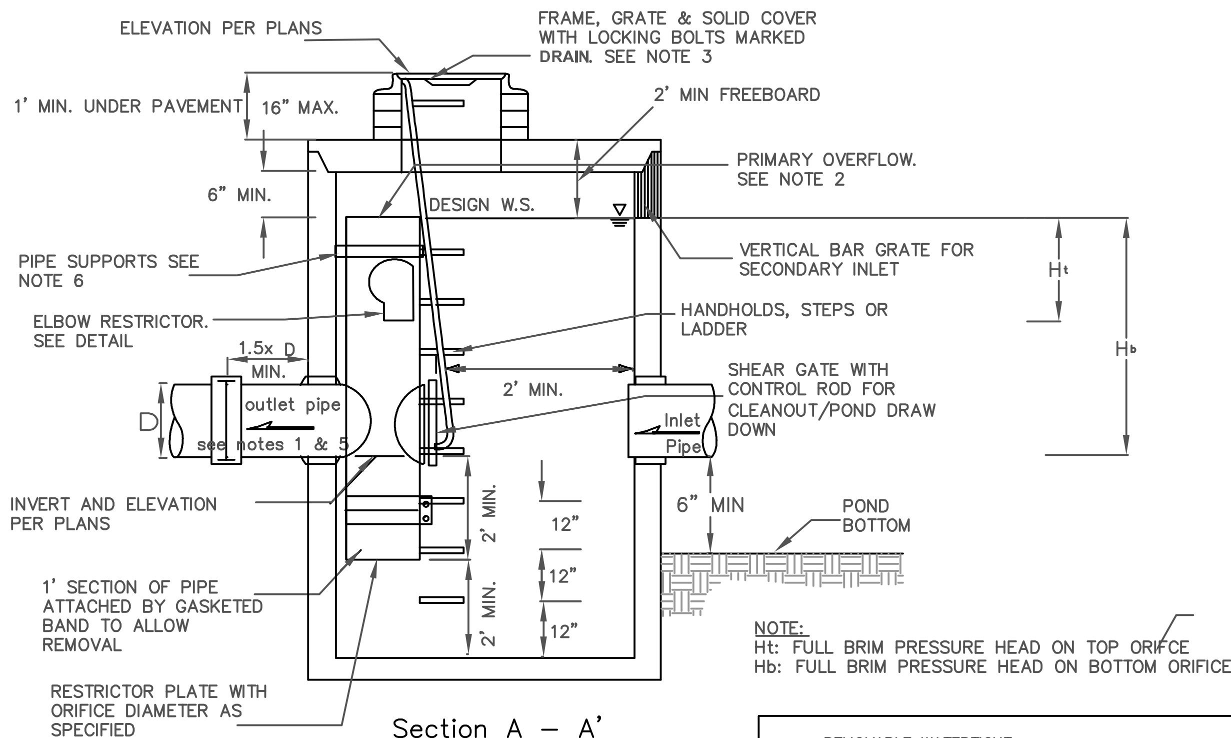
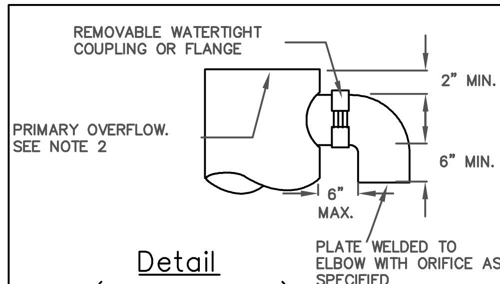
Figure 6-16: Perforated Riser Outlet



Plan View
(Not to Scale)



Section A – A'
(Not to Scale)



Detail
(Not to Scale)

NOTES:

1. USE A MINIMUM OF A 54" DIA TYPE 2 CATCH BASIN
2. OUTLET CAPACITY: 100-YEAR PEAK FLOW FOR ON-LINE BASINS.
3. METAL PARTS: CORROSION RESISTANT. NON-GALVANIZED PARTS PERFERRED. GALVANIZED PIPE PARTS TO HAVE ASPHALT TREATMENT.
4. FRAME AND LADDER OR STEPS OFFSET SO:
   A. CLEANOUT GATE IS VISIBLE FROM TOP.
   B. CLIMB-DOWN SPACE IS CLEAR OF RISER AND
   C. FRAME IS CLEAR OF CURB.
5. IF METAL OUTLET PIPE CONNECTS TO CEMENT CONCRETE PIPE: OUTLET PIPE TO HAVE SMOOTH O.D. EQUAL TO CONCRETE PIPE I.D. LESS ¼"
6. PROVIDE AT LEAST ONE 3 X .090 GAGE SUPPORT BRACKET ANCHORED TO CONCRETE WALL. (MAXIMUM 3' VERTICAL SPACING)
7. LOCATE ADDITIONAL LADDER RUNGS IN STRUCTURES USED AS ACCESS TO TANKS OR VAULTS TO ALLOW ACCESS WHEN CATCH BASIN IS FILLED WITH WATER

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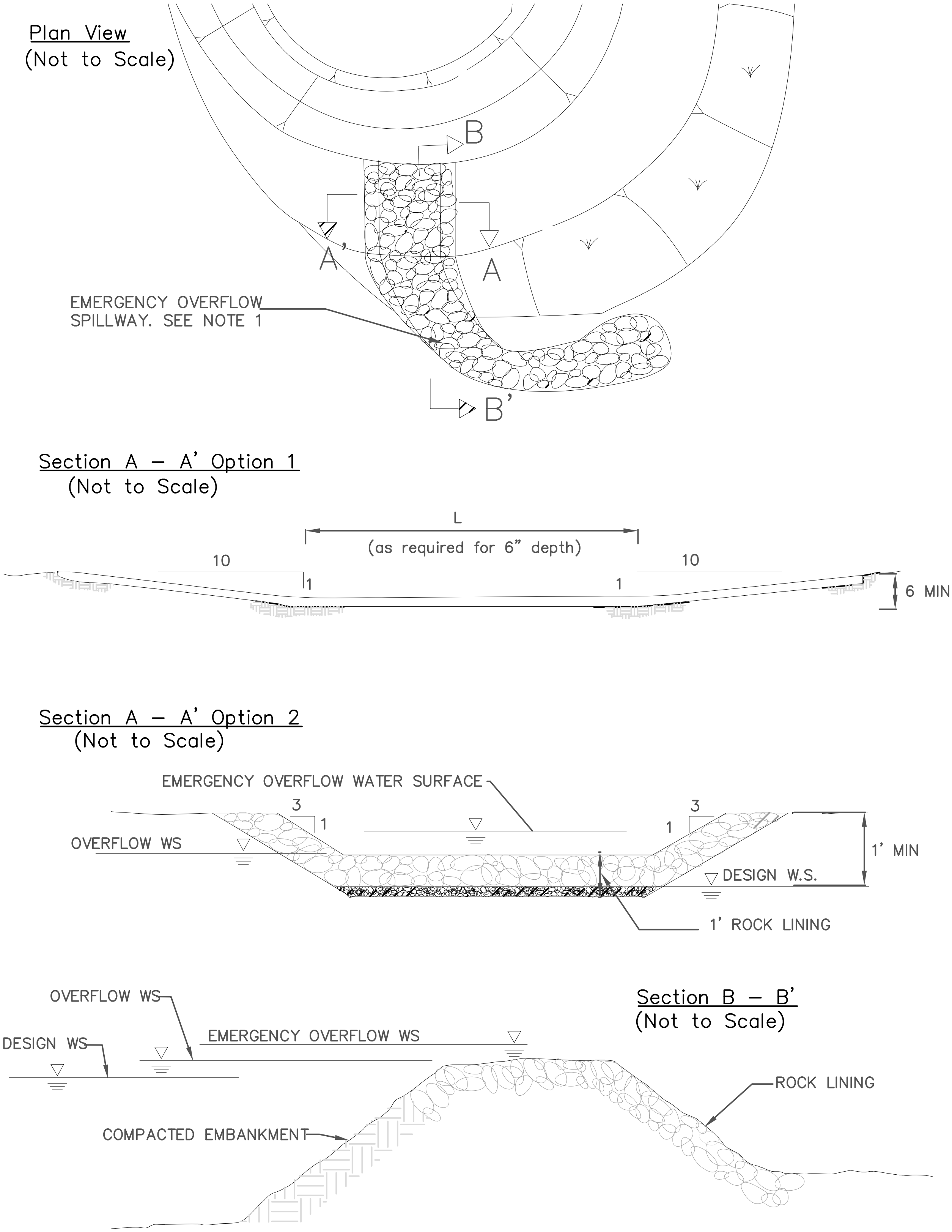
Figure 6-17: Multiple Orifice Outlet



NOTES:

1. ALTERNATIVE SPILLWAY DESIGNS BASED ON THE CALIFORNIA DEPARTMENT OF WATER RESOURCES' GUIDELINES FOR THE DESIGN AND CONSTRUCTION OF SMALL EMBANKMENT DAMS OR AT THE DISCRETION OF THE DEPARTMENT

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Figure 6-18: Spillway

Limitations

Limitations for dry extended detention basins include:

- Surface space availability - typically 0.5 to 2.0 percent of the total tributary development area required.
- Depth to groundwater - bottom of basin should be 2 feet higher than the seasonal high water table elevation.
- Steep slopes - basins placed above slopes greater than 15 percent or within 200 feet from the top of a hazardous slope or landslide area require a geotechnical investigation.
- Compatibility with flood control - basins must not interfere with flood control functions of existing conveyance and detention structures.

Design Criteria

Dry extended detention basins should be designed according to the requirements listed in Table 6-23 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-23: Dry Extended Detention Basin Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume (SQDV)	acre-feet	See Section 2 and Appendix E for calculating SQDV
Drawdown time for SQDV	hours	Top 50%: 12 hrs (minimum); Bottom 50%: 36 hrs
Basin Design Volume	acre-ft	1.2 * SQDV
Forebay basin size	acre-feet	5 to 15% of SQDV
Maximum forebay drain time	min	45
Low–flow channel depth	inches	9
Low-flow channel flow capacity		2*forebay outlet rate
Freeboard (minimum)	inches	12
Flow path length to width ratio	L:W	2:1, larger preferred; can be achieved using internal berms
Longitudinal slope	percentage	1 (forebay) and 0-2 (main basin)
Low flow channel geometry	feet	depth of 0.5 and width of 1
Minimum outflow device diameter	inches	18

Sizing Criteria

Dry extended detention (ED) basins are basins designed such that the SQDV is detained for 48 hours. This allows sediment particles and associated pollutants to settle and be removed from the stormwater. Procedures for sizing extended detention basins are summarized below. A sizing example is also provided.

Step 1: Calculate the design volume

Dry extended detention facilities shall be sized to capture and treat the SQDV (see Section E.1).

Step 2: Calculate the volume of the active basin

The total basin volume should be increased an additional 20% above the SQDV to account for sediment accumulation, at a minimum. If the basin is designed only for water quality treatment then the basin volume would be 120% of the SQDV. Freeboard is in additional to the total basin volume. Calculate the volume of the active basin (ft^2) (V_a):

$$V_a = 1.20*SQDV \qquad \text{(Equation 6-37)}$$

Step 3: Determine detention basin location and preliminary geometry based on site constraints

Based on site constraints, determine the basin geometry (area and length) and the storage available by developing an elevation-storage relationship for the basin. The cross-sectional geometry across the width of the basin should be approximately trapezoidal. Shallow side slopes are necessary if the basin is designed to have recreational uses during dry weather conditions.

1) Calculate the width of the basin footprint (W_{tot}) as follows:

$$W_{tot} = \frac{A_{tot}}{L_{tot}} \qquad \text{(Equation 6-38)}$$

Where:

A_{tot} = total surface area of the basin footprint (ft^2)

L_{tot} = total length of the basin footprint (ft)

2) Calculate the length of the active volume surface area including the internal berm but excluding the freeboard, ($L_{av\text{-}tot}$):

$$L_{av-tot} = L_{tot} - 2Zd_{fb} \qquad \text{(Equation 6-39)}$$

Where:

Z	=	interior side slope as length per unit height (H:V)
d_{fb}	=	freeboard depth (ft)

3) Calculate the width of the active volume surface area including the internal berm but excluding freeboard (ft), ($W_{av\text{-}tot}$):

$$W_{av-tot} = W_{tot} - 2Zd_{fb}$$ (Equation 6-40)

4) Calculate the total active volume surface area including the internal berm and excluding freeboard, ($A_{av\text{-}tot}$):

$$A_{av-tot} = L_{av-tot} \times W_{av-tot}$$ (Equation 6-41)

5) Calculate the area of the berm, (A_{berm}):

$$A_{berm} = W_{berm} \times L_{berm}$$ (Equation 6-4243)

Where:

W_{berm}	=	width of the internal berm
L_{berm}	=	length of the internal berm (= width excluding freeboard, $W_{av\text{-}tot}$)

6) Calculate the surface area excluding the internal berm and freeboard, A_{av}:

$$A_{av} = A_{av = tot} - A_{berm}$$ (Equation 6-44)

Step 4: Determine Dimensions of Forebay

The forebay should be sized to at least 5 to 15% of the basin active volume (V_a). Calculate the active volume of the forebay, (V_1):

$$V_1 = \frac{V_a \times \%V_1}{100}$$ (Equation 6-45)

Where:

$\%V_1$	=	percent of V_a in forebay (%)
V_a	=	total active volume (ft^3)

7) Calculate the surface area for the active volume of forebay (A_1):

$$A_1 = \frac{V_1}{d_1}$$ (Equation 6-46)

Where:

d_1 = average depth for the forebay (ft)

8) Calculate the length of forebay, (L_1):

$$L_1 = \frac{A_1}{W_1} \qquad \text{(Equation 6-47)}$$

Where:

W_1 = width of forebay (ft)

Step 5: Determine Dimensions of Cell 2

Cell 2 will consist of the remainder of the basin's active volume.

1) Calculate the active volume of Cell 2, (V_2):

$$V_2 = V_a - V_1 \qquad \text{(Equation 6-48)}$$

Where:

V_a = total basin active volume (ft^3)

V_1 = volume of forebay (ft^3)

2) Calculate the surface area, A_2, for the active volume of Cell 2:

$$A_2 = A_{av} - A_1 \qquad \text{(Equation 6-49)}$$

Where:

A_{av} = basin surface area excluding berm and freeboard (ft^2)

A_1 = surface area of forebay (ft^2)

3) Calculate the average depth (d_2) for the active volume of Cell 2:

$$d_2 = \frac{V_2}{A_2} \qquad \text{(Equation 6-50)}$$

4) Calculate the length of Cell 2, (L_2):

$$L_2 = \frac{A_2}{W_2} \qquad \text{(Equation 6-51)}$$

Where:

W_2 = width of Cell 2 (ft)

5) Verify that the length-to-width ratio of Cell 2 at half of d_2 is at least 1.5:1 with ≥2:1 preferred. If the length-to-width ratio is less than 1.5:1, modify input parameters until a ratio of at least 1.5:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the basin should be chosen. Calculate the length-to width (LW_{mid2}) ratio of Cell 2 at half of d_2 follows:

$$LW_{mid2} = \frac{L_{mid2}}{W_{mid2}} \quad \text{(Equation 6-52)}$$

Where:

W_{mid2} = $W_2 - Zd_2$ (Equation 6-53)

L_{mid2} = $L_2 - Zd_2$ (Equation 6-54)

W_{mid2} = width of Cell 2 at half of d_2 (ft)

L_{mid2} = length of Cell 2 at half of d_2 (ft)

Z = interior side slope as length per unit height (H:V)

d_2 = cell 2 average depth (ft)

Step 6: Ensure Design Requirements and Site Constraints are achieved

Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location or alternative treatment BMP.

Step 7: Size Outlet Structure

The total drawdown time for the basin should be 48 hours. The outlet structure should be designed to release the bottom 50% of the detention volume (half-full to empty) over 36 hours, and the top half (full to half-full) in 12 hours. A primary overflow should be sized to pass the peak flow rate from the developed capital design storm. See Section 6 for outlet structure sizing methodologies.

Step 8: Determine Emergency Spillway Requirements

For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm in order to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the 100-yr, 24-hr post-development peak stormwater runoff discharge rate directly to the downstream conveyance system or another acceptable discharge point. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

Sizing and Geometry

1) The total basin volume should be increased an additional 20% of the SQDV to account for sediment accumulation, at a minimum. If the basin is designed only for water quality treatment then the basin volume would be 120% of the SQDV. Freeboard is in addition to the total basin volume.

2) The minimum freeboard should be at least 1 foot above the emergency overflow water surface for dry extended detention basins.

3) The minimum flow-path length to width ratio at half basin height should be a minimum of 3:1 (L:W) and can be achieved using internal berms or other means to prevent short-circuiting. Intent: a long flow length will improve fine sediment removal.

4) The cross-sectional geometry across the width of the basin should be approximately trapezoidal. Shallow side slopes are necessary if the basin is designed to have recreational uses during dry weather conditions.

5) All dry ED basins should be free draining and a low flow channel should be provided. A low flow channel is a narrow, shallow trench filled with pea gravel and encased with filter fabric that runs the length of the basin to drain dry weather flows. The low flow channel should be of sufficient size considering the natural characteristics of the soil and have a positive-draining gradient flowing toward the outlet structure (typically 1 ft wide by 6 inches deep). If infiltration rates of subsurface soils are insufficient, the low flow channel should tie into perforated pipe at the outlet structure. If a sand filter or planting media is provided beneath the dry ED basin for increased volume reduction, it may be designed to take the place of the low flow channel.

6) The basin bottom should have a 1% longitudinal slope (direction of flow) in the forebay, and may range from 0 to 2% longitudinal slope in the main basin. The bottom of the basin should slope 2% toward the center low flow channel.

7) A basin should be large enough to allow for equipment access via a graded ramp.

Soils Considerations

1) The slopes of the detention basin should be analyzed for slope stability using rapid drawdown conditions and should meet the minimum standards set by the Ventura County Flood Control District. A 1.5 static factor of safety should be used. Seismic analysis is not required due to the temporary storage of water in the basin.

2) The infiltration capability of the dry ED basin can be enhanced by incorporating soil amendments.

Energy Dissipation

1) Energy dissipation controls constructed of sound materials such as stones, concrete, or proprietary devices that are rated to withstand the energy of the influent flow should be installed at the inlet to the sediment forebay. Flow velocity into the basin forebay should be controlled to 4 feet per second (ft/sec) or less.

2) Energy dissipation controls must also be used at the outlet/spillway from the detention basin unless the basin discharges to a storm drain or hardened channel.

Sediment Forebay

As untreated stormwater enters the dry ED basin, it passes through a sediment forebay for coarse solids removal. The forebay may be constructed using an internal berm constructed out of earthen embankment material, grouted riprap, stop logs, or other structurally sound material.

1) The basin should be sized so that 5 to 15% of the total basin volume is in the forebay and 85 to 95% of the total basin volume is in the main portion of the basin.

2) A gravity drain outlet from the forebay (2 inch minimum diameter) should extend the entire width of the internal berm and be designed to completely drain to the main basin within 10 minutes.

3) The forebay outlet should be offset (horizontally) from the inflow streamline to prevent short-circuiting.

4) Permanent steel post depth markers should be placed in the forebay to define sediment removal limits at 50% of the forebay sediment storage depth.

Vegetation

Vegetation within the dry ED basin provides erosion protection from wind and water and biofiltration of stormwater. The local permitting authority should review and approve any proposed basin landscape plan prior to implementation and following guidelines should be followed:

1) The bottom and slopes of the dry ED basin should be vegetated. A mix of erosion-resistant plant species that effectively bind the soil should be used on the slopes and a diverse selection of plants that thrive under the specific site, climatic, and watering conditions should be specified for the basin bottom. The basin bottom should not be planted with trees, shrubs, or other large woody plants that may interfere with sediment removal activities. The basin should be free of floating objects. Only native perennial grasses, forbs, or similar vegetation that can be replaced via seeding should be used on the basin bottom.

    a. Landscaping outside of the basin is required for all dry ED basins and should adhere to the following criteria so as not to hinder maintenance operations:

b. No trees or shrubs may be planted within 15 feet of inlet or outlet pipes or manmade drainage structures such as spillways, flow spreaders, or earthen embankments. Species with roots that seek water, such as willow or poplar, should not be used within 50 feet of pipes or manmade structures. Weeping willow (Salix babylonica) should not be planted in or near detention basins.

2) Prohibited non-native plant species will not be permitted. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website- or the California Invasive Plant Council website at www.cal-ipc.org.

3) A plant list provided by a landscape professional should be used as a guide only and should not replace project-specific planting recommendations, including recommendations on appropriate plants, fertilizer, mulching applications, and irrigation requirements (if any) to ensure healthy vegetation growth.

Sand Filter or Planting Media Layer

For increasing the volume reduction capability of a dry ED basin, an appropriately sized sand filter or planting media layer can be placed beneath the dry ED basin to achieve desired volume reduction goals if soil and slope conditions allow (i.e., infiltration rate greater than 0.5 in/hr but less than 2.4 in/hr; site slope less than 15%). The drawdown time of the sand filter or planting media layer should be less than 72 hours. The base of the sand filter or planting media layer should be level (i.e., zero slope). If a sand filter/planting media layer is provided over the length of the basin, it can take the place of the low-flow channel so long as it is designed to adequately infiltrate dry weather flows. Sizing of the sand filter and planting media layer for dry ED basins is the same as for sand filters and bioretention areas, respectively. The depth of water in the dry ED basin should not exceed 6 feet.

Outlet Structure and Drawdown Time

A drawdown time of 36 to 48 hours shall be provided for the SQDV. This drawdown time is for the volume in the basin above the sand filter layer (if provided) and serves the purpose of water quality treatment. An outflow device should be designed to release the bottom 50% of the detention volume (half-full to empty) over 24 to 32 hours, and the top half (full to half-full) in 12 to 16 hours. *The intention is that the drawdown schemes that detain low flows for longer periods than high flows have the following advantages over outlets that drain the basin evenly:*

- Greater flood control capabilities
- Enhanced treatment of low flows which make up the bulk of incoming flows.

Additional storage, detention, and outlet control is required to achieve pre-development stormwater runoff discharge rates for hydromodification control. The outlet structure can be designed to achieve flow control for meeting the multiple objectives of water quality and flow attenuation.

The outflow device (i.e., outlet pipe) should be oversized (18 inch minimum diameter). There are two options that can be used for the outlet structure:

1) Uniformly perforated riser structures.

2) Multiple orifice structures (orifice plate).

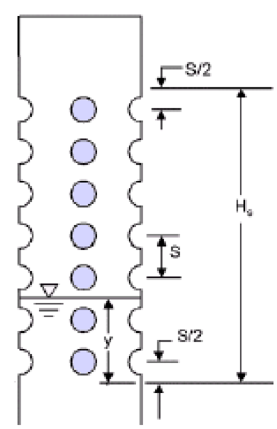
The outlet structure can be placed in the basin with a debris screen (Figure 6-15) or housed in a standard manhole (Figure 6-16). If a multiple orifice structure is used, an orifice restriction (if necessary) should be used to limit orifice outflow to the maximum discharge rates allowable for achieving the desired water quality and flow control objectives. Orifice restriction plates should be removable for emergency situations. A removable trash rack should be provided at the outlet.

Note that a primary overflow (typically a riser pipe connected to the outlet works) should be sized to pass flows larger than the stormwater quality design storm (if the ED basin is sized only for water quality) or to pass flows larger than the peak flow rate of the maximum design storm to be detained in the basin (e.g., 100-yr, 24-hr). The primary overflow is intended to protect against overtopping or breaching of a basin embankment.

Perforated Risers Outlet Sizing Methodology

The following attributes influence the perforated riser outlet sizing calculations**:**

- Shape of the basin (e.g., trapezoidal)
- Depth and volume of the basin
- Elevation / depth of first row of holes
- Elevation / depth of last row of holes
- Size of perforations
- Number of rows or perforations and number of perforations per row
- Desired drawdown time (e.g., 16 hour and 32 hour draw down for top half and bottom half respectively, 48 hour total drawdown time for the stormwater quality design volume)



Perforated Riser Outlet

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The governing rate of discharge from a perforated riser structure can be calculated using Equation 6-44 below:

$$Q = C_p \frac{2A_p}{3H_s} \sqrt{2g} H^{3/2}$$

(Equation 6-55)

Where:

Q	=	riser flow discharge (cfs)
C_p	=	discharge coefficient for perforations (use 0.61)
A_p	=	cross-sectional area of all the holes (ft^2)
s	=	center to center vertical spacing between perforations (ft)
H_s	=	distance from s/2 below the lowest row of holes to s/2 above the top row of holes (McEnroe 1988).
H	=	effective head on the orifice (measured from center of orifice to water surface)

For the iterative computations needed to size the perforations in the riser and determine the riser height, a simplified version of Equation 6-44 may be used as shown below in Equation 6-45 and Equation 6-46:

$$Q = kH^{3/2}$$

(Equation 6-56)

Where:

H	=	effective head on the orifice (measured from center of orifice to water surface)

$$k = C_p \frac{2A_p}{3H_s} \sqrt{2g}$$

(Equation 6-57)

Where:

C_p	=	discharge coefficient for perforations (use 0.61)
A_p	=	cross-sectional area of all the holes (ft^2)
s	=	center to center vertical spacing between perforations (ft)

H_S	=	distance from s/2 below the lowest row of holes to s/2 above the top row of holes.
g	=	32.17 ft/sec^2

Uniformly perforated riser designs are defined by the depth or elevation of the first row of perforations, the length of the perforated section of pipe, and the size or diameter of each perforation.

Multiple Orifice Outlet Sizing Methodology

The following attributes influence multiple orifice outlet sizing calculations**:**

- Shape of the basin (e.g., trapezoidal)
- Depth and volume of the basin
- Elevation of each orifice
- Desired draw-down time (e.g., 16 hour and 32 hour draw down times for top half and bottom half respectively, 48 hour drawdown time for stormwater quality design volume)

The rate of discharge from a single orifice can be calculated using Equation 6-22.

$$Q = CA(2gH)^{0.5}$$ (Equation 6-58)

Where:

Q	=	orifice flow discharge
C	=	discharge coefficient
A	=	cross-sectional area of orifice or pipe (ft2)
g	=	acceleration due to gravity (32.2 ft/s2)
H	=	effective head on the orifice (measured from center of orifice to water surface)

Multiple orifice designs are defined by the depth (or elevation) and the size (or diameter) of each orifice. The steps needed to size a dual orifice outlet are outlined in Appendix E; multiple orifices may be provided and sized using a similar approach.

Emergency Spillway

An emergency overflow spillway in addition to the primary overflow outlet (as described above) is required. The emergency spillway should be sized for flows greater than the peak 100-year 24-hour storm if the basin is designed on-line or, if the basin is designed on-line, the spillway should be sized for flows greater than the basin design volume (e.g., stormwater quality design volume). The spillway should provide for adequate energy dissipation downstream. The spillway should allow for at least 12 inches of freeboard above the emergency overflow water surface elevation if the basin is on-line. If the basin is on-line, 2 feet of freeboard is preferable.

Spillways shall meet the California Department of Water Resources, Division of Safety of Dams Guidelines for the Design and Construction of Small Embankment Dams (http://damsafety.water.ca.gov/docs/GuidelinesSmallDams.pdf). *Intent: Emergency overflow spillways are intended to control the location of basin overtopping and safely direct overflows back into the downstream conveyance system or other acceptable discharge point.*

On-line Basins

1) On-line basins must have an emergency overflow spillway to prevent overtopping of walls or berms should blockage of the primary outlet occur based on a downstream risk assessment.

2) The overflow spillway must be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm.

3) The minimum freeboard should be 1 foot (but preferably at least 2 feet) above the maximum water surface elevation over the emergency spillway.

Off-line Basins

1) Off-line basins must have either an emergency overflow spillway or an emergency overflow riser. The emergency overflow must be designed to pass the 100-yr 24-hr post-development peak stormwater runoff discharge rate directly to the downstream conveyance system or another acceptable discharge point. Where an emergency overflow spillway would discharge to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

2) The emergency overflow spillway shall be armored to withstand the energy of the spillway flows.

3) The minimum freeboard should be 1 foot above the maximum water surface elevation over the emergency spillway.

Side Slopes

1) Interior side slopes above the stormwater quality design depth and up to the emergency overflow water surface steeper than 4:1 (H:V) should be stabilized to prevent erosion with a method approved by the local permitting authority.

2) Exterior side slopes steeper than 2:1 (H:V) should be stabilized to prevent erosion with a method approved by the local permitting authority.

3) For any slope (interior or exterior) greater than 2:1 (H:V), a geotechnical investigation and report must be submitted and approved by the local permitting authority.

4) Landscaped slopes should be no greater than 3:1 (H:V) to allow for maintenance.

5) Basin walls may be vertical retaining walls, provided: (a) they are constructed of reinforced concrete, (b) a fence is provided along the top of the wall (see fencing below) or further back, and (c) the design is stamped by a licensed civil engineer and approved by the Local permitting authority.

Embankments

1) Earthworks and berm embankments should be performed in accordance with the latest edition of the "Greenbook Standard Specifications for Public Works Construction".

2) Embankments are earthen slopes or berms used for detaining or redirecting the flow of water.

3) Top of berm separating forebay and main basin should be 2 feet minimum below the stormwater quality design water surface and should be keyed into embankment a minimum of 1 foot on both sides.

4) Typically, the top width of berm embankments are at least 20 feet, but narrower embankments may be plausible if approved by the civil engineer and the Local permitting authority.

5) Basin berm embankments should be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a licensed civil engineer) free of loose surface soil materials, roots, and other organic debris.

6) The berm embankment should be constructed of compacted soil (95% minimum dry density, modified proctor method per ASTM D1557), placed in 6-inch lifts.

7) Basin berm embankments greater than 4 feet in height should be constructed by excavating a key equal to 50% of the berm embankment cross-sectional height and width. This requirement may be waived if specifically recommended by a licensed civil engineer.

8) The berm embankment should be constructed of compacted soil (95% minimum dry density, modified proctor method per ASTM D1557), placed in 6-inch lifts.

9) Low growing native or non-invasive perennial grasses should be planted on downstream embankment slopes. See vegetation section below.

Fencing

1) Safety is provided either by fencing of the facility or by managing the contours of the basin to eliminate drop-offs and other hazards.

2) If fences are required, fences should be designed and constructed in accordance with relevant standards and should typically be located at or above the overflow water surface elevation. Shrubs (approved, California-adapted species) can be used to hide the fencing. See vegetation section above.

Right-of-Way

1) Dry extended detention basins and associated access roads to be maintained by a public agency should be dedicated in fee or in an easement to the public agency with appropriate access.

Maintenance Access

1) Ownership of the basin and maintenance thereof is the responsibility of the developer/applicant. A maintenance agreement with the Local permitting authority is required to ensure adequate performance and allow emergency access to the facilities.

2) Maintenance access road(s) should be provided to the control structure and other drainage structures associated with the basin (e.g., inlet, emergency overflow or bypass structures). Manhole and catch basin lids should be in or at the edge of the access road.

3) A ramp into the basin should be constructed near the basin outlet. An access ramp is required for removal of sediment with a backhoe or loader and truck. The ramp should extend to the basin bottom to avoid damage to vegetation planted on the basin slope.

4) All access ramps and roads should be provided in accordance with the current policies of the Ventura County Flood Control District or local approval authority.

Construction Considerations

The use of treated wood or galvanized metal anywhere inside the facility is prohibited.

Operations and Maintenance

Maintenance is of primary importance if extended detention basins are to continue to function as originally designed. A maintenance agreement must be developed with the local approval authority to ensure adequate performance and allow emergency access. Maintenance of the basin is the responsibility of the development, unless otherwise agreed upon.

A specific maintenance plan shall be formulated for each facility outlining the schedule and scope of maintenance operations, as well as the data handling and reporting requirements. The following are general maintenance requirements:

1) The basin should be inspected semiannually or more frequently, and inspections after major storm events are encouraged (see Appendix I for guidance on facility maintenance inspections). Trash and debris should be removed as needed, but at least annually prior to the beginning of the wet season (see Appendix I for dry extended detention basin inspection and maintenance checklist).

2) Site vegetation should be maintained as follows:

    - Vegetation, large shrubs, or trees that limit access or interfere with basin operation should be pruned or removed.
    - Slope areas that have become bare should be revegetated and eroded areas should be regraded prior to being revegetated.
    - Grass should be mowed to 4 to 9 inch high and grass clippings should be removed.
    - Fallen leaves and debris from deciduous plant foliage should be raked and removed.
    - Invasive vegetation, such as Alligatorweed (Alternanthera philoxeroides), Halogeton (Halogeton glomeratus), Spotted Knapweed (Centaurea maculosa), Giant Reed (Arundo donax), Castor Bean (Ricinus communis), Perennial Pepperweed (Lepidium latifolium), and Yellow Starthistle (Centaurea solstitalis) should be removed and replaced with non-invasive species. Invasive species should never contribute more than 25% of the vegetated area. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website or the California Invasive Plant Council website at www.cal-ipc.org.
    - Dead vegetation should be removed if it exceeds 10% of area coverage. Vegetation should be replaced immediately to maintain cover density and control erosion where soils are exposed.
    - No herbicides or other chemicals should be used to control vegetation.

3) Sediment buildup exceeding 50% of the forebay capacity should be removed. Sediment from the remainder of the basin should be removed when 6 inches of sediment accumulates. Sediments should be tested for toxic substance accumulation in compliance with current disposal requirements if land uses in the catchment include commercial or industrial zones, or if visual or olfactory indications of pollution are noticed. If toxic substances are encountered at concentrations exceeding thresholds of Title 22, Section 66261 of the California Code of Regulations,

the sediment must be disposed of in a hazardous waste landfill. It is recommended to clean the forebay frequently to reduce frequency of main basin cleaning.

4) Remove sediment from basin when accumulation reaches 25% of original design depth. Cleaning is recommended to occur in early spring to allow vegetation to reestablish.

5) Repair erosion to banks and bottom of basin as required.

6) Following sediment removal activities, replanting, and/or reseeding of vegetation may be required for reestablishment.

7) Control vectors as needed.

TCM-2: Wet Detention Basin

Wet detention basins are constructed, naturalistic ponds with a permanent or seasonal pool of water (also called a “wet pool” or “dead storage”). Aquascape facilities, such as artificial lakes, are a special form of wet pool facility that can incorporate innovative design elements to allow them to function as a stormwater treatment facility in addition to an aesthetic water feature. Wetponds require base flows to exceed or match losses through evaporation and/or infiltration and they must be designed with the outlet positioned and/or operated in such a way as to maintain a permanent pool. Wetponds can be designed to provide extended detention of incoming flows using the volume above the permanent pool surface.

Wet Detention Basin

Photo Credit: Geosyntec Consultants

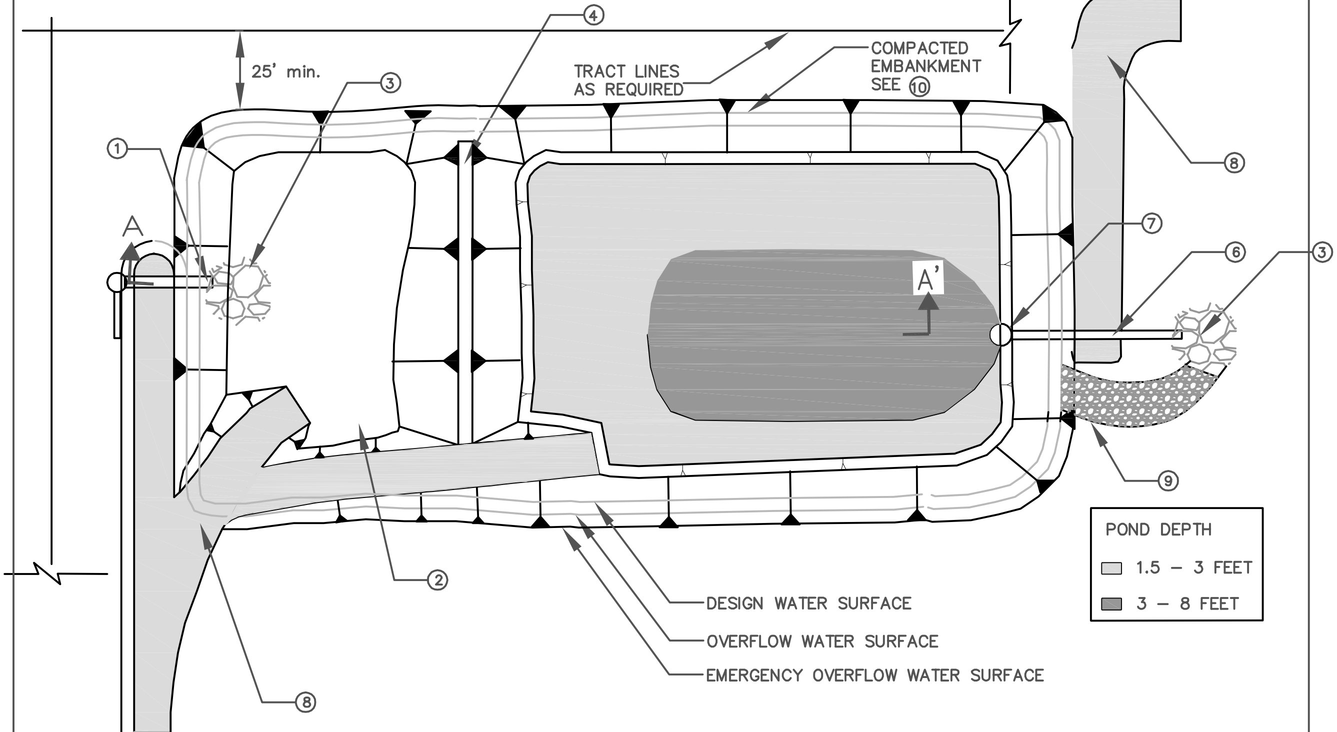
Application

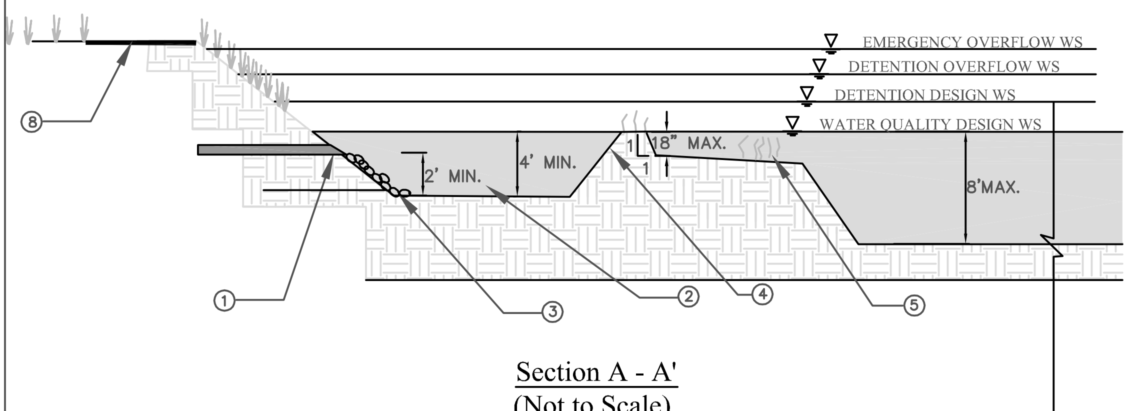
- Regional detention & treatment
- Roads, highways, parking lots, commercial, residential
- Parks, open spaces, and golf courses

Preventative Maintenance

- inspected at a minimum annually and inspections after major storm events
- Pruned or remove vegetation, large shrubs, or trees that limit access or interfere with basin operation
- Remove sediment buildup at inlets and outlets

Plan View
(Not to Scale)





Section A - A'
(Not to Scale)

NOTES:

1. INLET PIPE SHOULD BE SUBMERGED WITH A MINIMUM OF 2' DISTANCE FROM THE BOTTOM
2. FIRST CELL VOLUME SHALL EQUAL 25% TO 35% OF TOTAL WETPOND VOLUME. DEPTH SHALL BE 4' MIN TO 8' MAX PLUS AN ADDITIONAL 1' MIN SEDIMENT STORAGE DEPTH.
3. RIP RAP APRON OR OTHER ENERGY DISSIPATION.
4. BERM SHALL EXTEND ACROSS ENTIRE WIDTH OF THE WETPOND.
5. EMERGENT VEGETATION SHALL BE PLANTED IN REGIONS OF THE POND THAT ARE 3' DEEP OR LESS.
6. SIZE OUTLET PIPE TO PASS 100-YEAR PEAK FLOW FOR ON-LINE PONDS AND WATER QUALITY PEAK FLOW FOR OFF-LINE PONDS.
7. WATER QUALITY OUTLET STRUCTURE. SEE FIGURE 8-2 AND FIGURE 8-3 FOR DETAILS.
8. MAINTENANCE ACCESS ROAD SHOULD PROVIDE ACCESS TO BOTH THE FIRST CELL AND MAIN BASIN.
9. INSTALL EMERGENCY OVERFLOW SPILLWAY AS NEEDED. SEE FIGURE 2-4 FOR DETAILS

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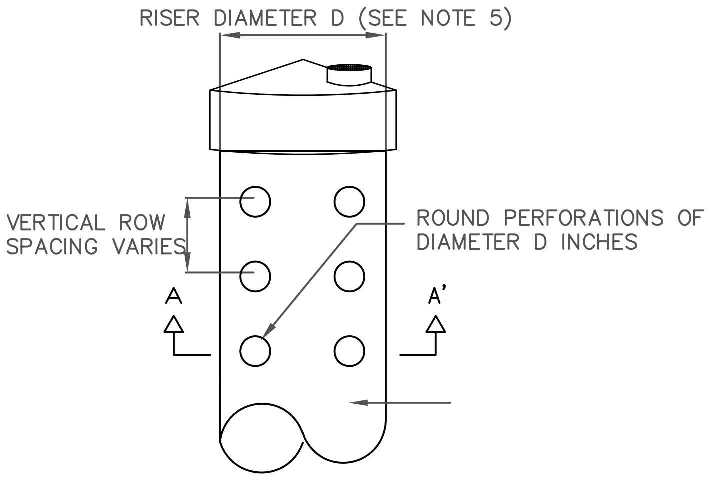
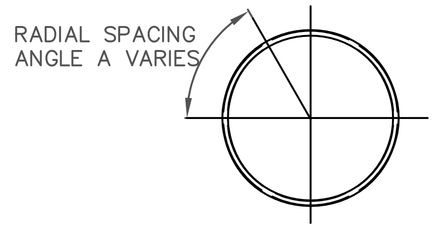
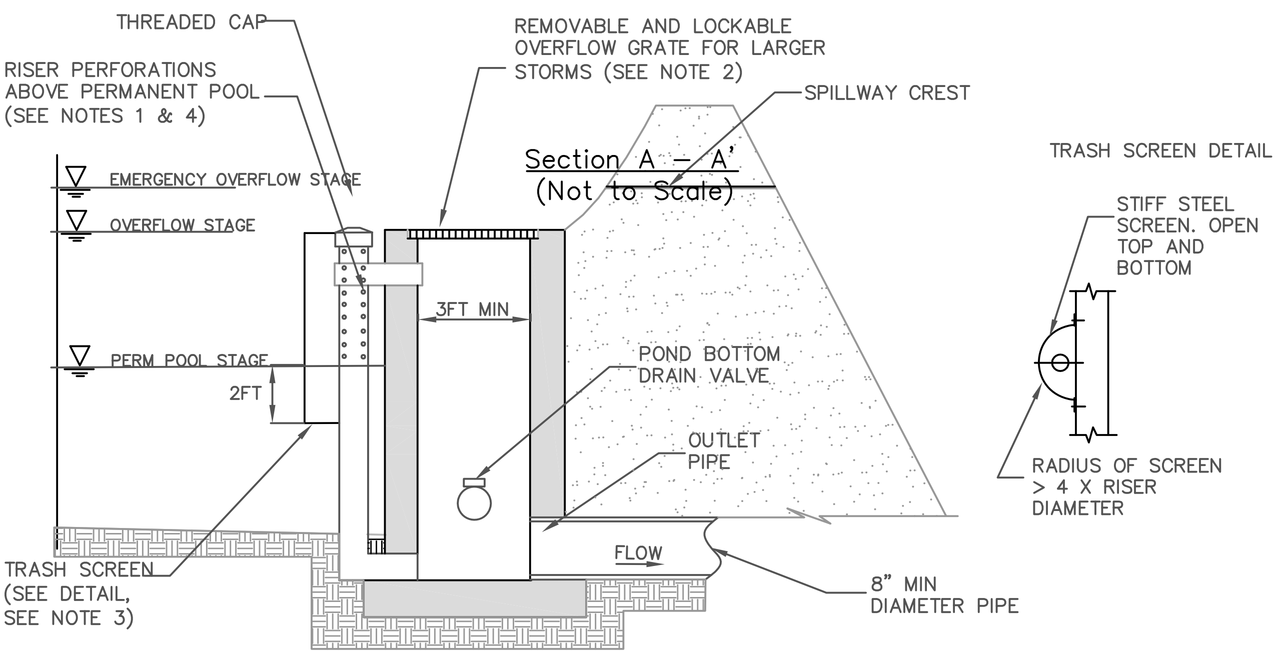
Figure 6-19: Wet Detention Basin



Perforated Riser Detail
(Not to Scale)



Section A – A'
(Not to Scale)



Perforated Riser Outlet Structure
(Not to Scale)

NOTES:

1. RISER PIPE SHALL BE SIZED TO PROVIDE 36 TO 48–HOUR FULL BRIM DRAW DOWN TIME.
2. TOTAL OUTLET CAPACITY: 100–YEAR PEAK FLOW FOR ON–LINE BASINS AND WATER QUALITY DESIGN FLOW FOR OFF–LINE BASINS.
3. SCREEN OPENINGS SHALL BE AT LEAST ¼" AND SHALL NOT EXCEED THE DIAMETER OF THE PERFORATIONS ON THE RISER.
4. RISER PIPE PERFORATION DIAMETER SHALL BE NO LESS THAN ½" AND NO MORE THAN 2"
5. MINIMUM PIPE DIAMETER (D) IS 2'
6. RISER PIPE MATERIAL IS CMP

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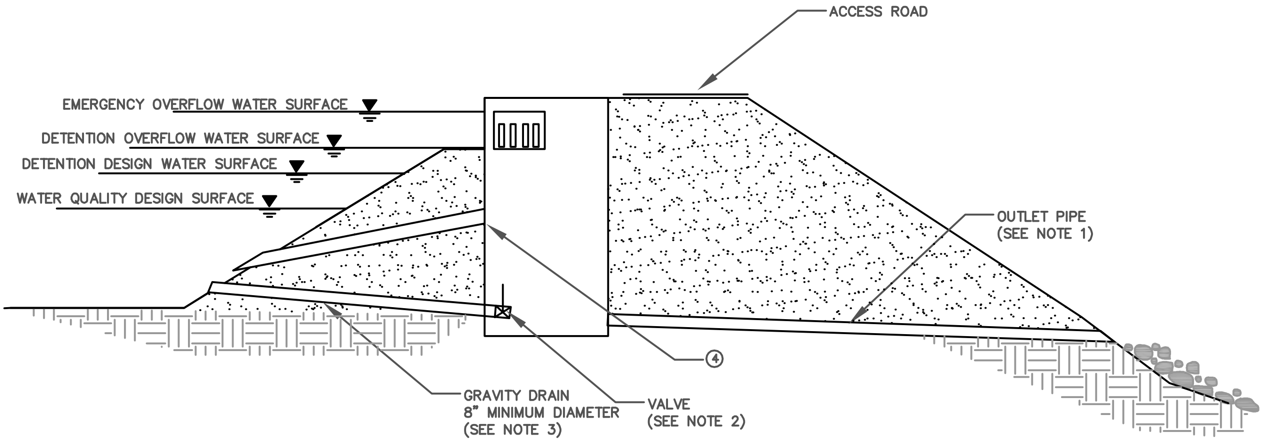
Figure 6-20: Riser Outlet



Inverted Pipe Outlet Structure
(Not to Scale)

NOTES:

1. SIZE OUTLET PIPE SYSTEM TO PASS 100–YEAR FLOW FOR ON–LINE PONDS AND WATER QUALITY PEAK FLOW FOR OFF–LINE PONDS.
2. VALVE MAY BE LOCATED INSIDE MANHOLE OR OUTSIDE WITH APPROVED OPERATIONAL ACCESS
3. INVERT OF DRAIN SHALL BE 6" MINIMUM BELOW TOP OF INTERNAL BERM. LOWER PLACEMENT IS DESIRABLE. INVERT SHALL BE 6" MINIMUM ABOVE BOTTOM OF POND.
4. OUTLET PIPE INVERT SHALL BE AT WETPOOL WATER SURFACE ELEVATION

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Figure 6-21: Inverted Pipe Outlet

Limitations

Limitations for wet detention basins include:

- Wet detention basins typically are used for treating areas larger than 10 acres and less than 10 square miles. They are especially applicable for regional water quality treatment and flow control.
- Off-line wet detention basins must not interfere with flood control functions of existing conveyance and detention structures.
- If wet detention basins are located in areas with site slopes greater than 15% or within 200 feet of a hazardous steep slope or mapped landslide area (on the uphill side), a geotechnical investigation and report must be provided to ensure that the basin does not compromise the stability of the site slope or surrounding slopes.
- Wet detention basins require a regular source of base flow if water levels are to be maintained. If base flow is insufficient during summer months, supplemental water may be necessary to maintain water levels.

Design Criteria

The main challenge associated with wet detention basins is maintaining desired water levels. A wet detention basin should be designed according to the requirements listed in Table 6-24 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-24: Wet Detention Basin Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume, SQDV	acre-ft	See Section 2 and Appendix E for calculating SQDV.
Permanent Pool Volume		SQDV
Forebay Volume		5 to 10% of SQDV
Maximum Forebay Drain Time	min	45
Depth without sediment storage	feet	0.5-12 (littoral zone, 25-40% permanent pool) 4 (first cell minimum) 8 (any cell maximum) Deeper zone: 4-8 feet average; 12 feet maximum depth
Maximum residence time	Days	7 (dry weather)
Freeboard (minimum)	inches	12

Flow path length to width ratio	L:W	2:1 (larger preferred)
Side slope (maximum)	H:V	4:1 (H:V) Interior and 3:1 (H:V) Exterior
Longitudinal slope	percentage	1 (forebay) and 0-2 (main basin)
Vegetation Type	--	Varies see vegetation section below
Vegetation Height	--	Varies see vegetation section below
Buffer zone (minimum)	feet	25
Minimum outflow device diameter	inches	18

Sizing Criteria

Wet Detention basins may be designed with or without extended detention above the permanent pool. The extended detention portion of the wet detention basin above the permanent pool, if provided, functions like a dry extended detention (ED) basin (see VEG-5: Dry Extended Detention Basin). If there is no extended detention provided, wet detention basins shall be sized to provide a minimum wet pool volume equal to the stormwater quality design volume plus an additional 5% for sediment accumulation. If extended detention is provided above the permanent pool, the sizing is dependent of the functionality of the basin; the basin may function as water quality treatment only or water quality plus peak flow attenuation.

If the basin is designed for water quality treatment only, then the permanent pool volume should be a minimum of 10 percent of the stormwater quality design volume and the surcharge volume (above the permanent pool) should make up the remaining 90 percent. If extended detention is provided above the permanent pool and the basin is designed for water quality treatment and peak flow attenuation, then the permanent pool volume should be equal to the water quality treatment volume, and the surcharge volume should be sized to attenuate peak flows in order to meet the peak runoff discharge requirements. The extended detention portion of the wet detention basin above the permanent pool, if provided, functions like a dry extended detention (ED) basin (see VEG-5: Dry Extended Detention Basin).

Step 1: Calculate the design volume

Wet detention basins shall be sized with a permanent pool volume equal to the SQDV volume (see Section 2 and Appendix E).

Step 2: Determine the active design volume for the wet detention basin without extended detention

The active volume of the wet detention basin, V_a, shall be equal to the SQFV plus an additional 5% for sediment accumulation.

$$V_a = 1.05 \times SQDV \qquad \text{(Equation 6-59)}$$

Step 3: Determine pond location and preliminary geometry based on site constraints

Based on site constraints, determine the pond geometry and the storage available by developing an elevation-storage relationship for the pond. Note that a more natural geometry may be used and is in many cases recommended; the preliminary basin geometry calculations should be used for sizing purposes only.

1) Calculate the width of the pond footprint, W_{tot}, as follows:

$$W_{tot} = \frac{A_{tot}}{L_{tot}} \qquad \text{(Equation 6-60)}$$

Where:

A_{tot} = total surface area of the pond footprint (ft^2)

L_{tot} = total length of the pond footprint (ft)

1) Calculate the length of the active volume surface area including the internal berm but excluding the freeboard, $L_{av\text{-}tot}$:

$$L_{av-tot} = L_{tot} - 2Zd_{fb} \qquad \text{(Equation 6-61)}$$

Where:

Z = interior side slope as length per unit height

d_{fb} = freeboard depth

2) Calculate the width of the active volume surface area including the internal berm but excluding freeboard, $W_{av\text{-}tot}$:

$$W_{av-tot} = W_{tot} - 2Zd_{fb} \qquad \text{(Equation 6-62)}$$

3) Calculate the total active volume surface area including the internal berm and excluding freeboard, $A_{av\text{-}tot}$:

$$A_{av-tot} = L_{av-tot} \times W_{av-tot} \qquad \text{(Equation 6-63)}$$

4) Calculate the area of the berm, A_{berm}:

$$A_{berm} = W_{berm} \times L_{berm} \qquad \text{(Equation 6-64)}$$

Where:

W_{berm} = width of the internal berm

L_{berm} = length of the internal berm

5) Calculate the active volume surface area excluding the internal berm and freeboard, A_{wq}:

$$A_{wq} = A_{wq\ =\ tot} - A_{berm} \qquad \text{(Equation 6-65)}$$

Step 4: Determine Dimensions of Forebay

The wet detention basin should be divided into two cells separated by a berm or baffle. The forebay should contain between 5 and 10 percent of the total volume. The berm or baffle volume should not count as part of the total volume. Calculate the active volume of forebay, V_1:

$$V_1 = \frac{V_a \times \%V_1}{100} \qquad \text{(Equation 6-66)}$$

Where:

$\%V_1$ = percent of SQDV in forebay (%)

1) Calculate the surface area for the active volume of forebay, A_1:

$$A_1 = \frac{V_1}{d_1} \qquad \text{(Equation 6-67)}$$

Where:

d_1 = average depth fo rhte active volume of forebay (ft)

1) Calculate the length of forebay, L_1. Note, inlet and outlet should be configured to maximize the residence time.

$$L_1 = \frac{A_1}{W_1} \qquad \text{(Equation 6-68)}$$

Where:

W_1 = width of forebay (ft), $W_1 = W_{av\text{-}tot} = L_{berm}$

Step 5: Determine Dimensions of Cell 2

Cell 2 will consist of the remainder of the basin's active volume.

1) Calculate the active volume of Cell 2, V_2:

$$V_2 = V_a - V_1 \qquad \text{(Equation 6-69)}$$

2) The minimum wetpool surface area includes 0.3 acres of wetpool per acre-foot of permanent wetpool volume. Calculate A_{min2}:

$$A_{min2} = (V_2 \times 0.3 \ \frac{acres}{acre-feet}) \quad \text{(Equation 6-70)}$$

3) Calculate the actual wetpool surface area, A_2:

$$A_2 = A_{av} - A_1 \quad \text{(Equation 6-71)}$$

Verify that A_2 is greater than A_{min2}. If A_2 is less than A_{min2}, then modify input parameters to increase A_2 until it is greater than A_{min2}. If site constraints limit this criterion, then another site for the pond should be chosen.

4) Calculate the top length of Cell 2, L_2:

$$L_2 = \frac{A_2}{W_2} \quad \text{(Equation 6-72)}$$

Where:

W_2 = width of Cell 2 (ft), $W_2 = W_1 = W_{wq\text{-}tot} = L_{berm}$

5) Verify that the length-to-width ratio of Cell 2 is at least 1.5:1 with ≥ 2:1 preferred. If the length-to-width ratio is less than 1.5:1, modify input parameters until a ratio of at least 1.5:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the pond should be chosen.

$$LW_2 = \frac{L_2}{W_2} \quad \text{(Equation 6-73)}$$

6) Calculate the emergent vegetation surface area, A_{ev}:

$$A_{ev} = \frac{A_2 \bullet \%A_{ev}}{100} \quad \text{(Equation 6-74)}$$

Where:

$\%A_{ev}$ = percent of surface area that will be planted with emergent vegetation

7) Calculate the volume of the emergent vegetation shallow zone (1.5 – 3 ft), V_{ev}:

$$V_{ev} = A_{ev} \bullet d_{ev} \quad \text{(Equation 6-75)}$$

Where:

d_{ev} = average depth of the emergent vegetation shallow zone (1.5 – 3 ft)

8) Calculate the length of the emergent vegetation shallow zone, L_{ev}:

$$L_{ev} = \frac{A_{ev}}{W_{ev}} \qquad \text{(Equation 6-76)}$$

Where:

W_{ev} = width of the emergent vegetation shallow zone (ft), $W_{ev} = W_2$

9) Calculate the volume of the deep zone, V_{deep}:

$$V_{deep} = V_2 - V_{ev} \qquad \text{(Equation 6-77)}$$

10) Calculate the surface area of the deep (>3 ft) zone, A_{deep}:

$$A_{deep} = A_2 - A_{ev} \qquad \text{(Equation 6-78)}$$

11) Calculate the average depth of the deep zone (4-8 ft), d_{deep}:

$$d_{deep} = \frac{V_{deep}}{A_{deep}} \qquad \text{(Equation 6-79)}$$

12) Calculate length of the deep zone, L_{deep}:

$$L_{deep} = \frac{A_{deep}}{W_{deep}} \qquad \text{(Equation 6-80)}$$

Where:

W_{deep} = width of the deep zone (ft), $W_{deep} = W_2$

Step 6: Ensure design requirements and site constraints are achieved

Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location for the BMP.

Step 7: Size Outlet Structure

For extended detention wet detention basin, outlet structures should be designed to provide 12 to 48 hour emptying time for the water quality volume above the permanent pool.

The basin outlet pipe should be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for off-line basins or flows greater than the peak runoff discharge rate for the 100-year, 24-hr design storm for on-line basins.

Step 8: Determine Emergency Spillway Requirements

For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the water quality design storm. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

Sizing and Geometry

1) If there is no extended detention provided, wet detention basins shall be sized to provide a minimum wet pool volume equal to the stormwater quality design volume plus an additional 5% for sediment accumulation. If extended detention is provided above the permanent pool and the basin is designed for water quality treatment only, then the permanent pool volume should be a minimum of 10 percent of the stormwater quality design volume and the surcharge volume (above the permanent pool) should make up the remaining 90 percent. If extended detention is provided above the permanent pool and the basin is designed for water quality treatment and peak flow attenuation, then the permanent pool volume shall be equal to the water quality treatment volume and the surcharge volume should be sized to attenuate peak flows to meet the peak runoff discharge requirements. The extended detention portion of the wet detention basin above the permanent pool, if provided, functions like a dry extended detention (ED) basin (see TCM-1: Dry Extended Detention Basin).

2) The wet detention basin should be divided into two cells separated by a berm or baffle. The first cell should contain between 25 to 35 percent of the total volume. The berm or baffle volume should not count as part of the total volume. Intent: The full-length berm or baffle reduces short-circuiting and promotes plug flow.

3) Wet detention basins with wetpool volumes less than or equal to 4,000 cubic feet may be single-celled (i.e., no baffle or berm is required).

4) Sediment storage should be provided in the first cell. The sediment storage should have a minimum depth of 1 foot. This volume should not be included as part of the required water quality volume.

5) The minimum depth of the first cell should be 4 feet, exclusive of sediment storage requirements. The depth of the first cell may be greater than the depth of the second cell. Average depth should be between 4 feet and 8 feet.

6) For wet detention basin depths in excess of 6 feet, some form of recirculation should be provided, such as a fountain or aerator, to prevent stratification, stagnation and low dissolved oxygen conditions.

7) The edge of the basin should slope from the surface of the permanent pool to a depth of 12 to 18 inches at a slope of 1:1 or greater. If soil conditions will not support a 1:1 (H:V) slope then the steepest slope that can be supported should be used or a shallow retaining wall constructed (18 inch max). Beyond the edge of the basin, a bench sloped at 4:1 (H:V) maximum should extend into the basin to a depth of at least 3 feet. A steeper slope may be used beyond the 3 foot depth to a maximum of 8 feet. Intent: steep slopes at water's edge will minimize very shallow areas that can support mosquitoes.

8) At least 25% of the basin area should be deeper than 3 feet to prevent the growth of emergent vegetation across the entire basin. If greater than 50% of the wet pool area is in excess of 6 feet deep, some form of recirculation should be provided, such as a fountain or aerator, to prevent stratification, stagnation and low dissolved oxygen conditions.

9) A wet detention basin should have a surface area of not less than 0.3 acres for each acre-foot of permanent pool volume. In addition, extra area needed to provide a design that meets all other provisions of this section should be provided. Additional surface area in excess of the minimum may be provided. There is no maximum surface area provided that all provisions of this section are met.

10) Inlets and outlets should be placed to maximize the flowpath through the facility. The flowpath length-to-width ratio should be a minimum of 1.5:1, but a flowpath length-to-width ratio of 2:1 or greater is preferred. The flowpath length is defined as the distance from the inlet to the outlet, as measured at mid-depth. The width at mid-depth can be found as follows: width = (average top width + average bottom width)/2. Intent: a long flowpath length will improve fine sediment removal.

11) All inlets should enter the first cell. If there are multiple inlets, the length-to-width ratio should be based on the average flowpath length for all inlets.

12) The minimum freeboard should be 1 foot above the maximum water surface elevation (2 feet preferred) for on-line basins and 1 foot above the maximum water surface elevation for on-line basins.

13) The maximum residence time for dry weather flows should be 7 days. Intent: Vector control.

Internal Berms and Baffles

1) A berm or baffle should extend across the full width of the wet detention basin and be keyed into the basin side slopes. If the berm embankments are greater than 4 feet in height, the berm should be constructed by excavating a key equal to 50% of the embankment cross-sectional height and width. This requirement may be waived if recommended by a licensed civil engineer for the specific site conditions. The geotechnical investigation must consider the situation in which one of the two cells is empty while the other remains full of water.

2) The top of the berm should extend to the permanent pool surface or be one foot below the permanent pool surface to discourage public access. If the top of the berm is at the water permanent pool surface, the side slopes should be 4H:1V. Berm side slopes may be steeper (up to 3:1) if the berm is submerged one foot.

3) If good vegetation cover is not established on the berm, erosion control measures should be used to prevent erosion of the berm back-slope when the basin is initially filled.

4) The interior berm or baffle may be a retaining wall provided that the design is prepared and stamped by a licensed civil engineer. If a baffle or retaining wall is used, it should be submerged one foot below the permanent pool surface to discourage access by pedestrians.

5) Internal earthen berms 6 feet high or less should have a minimum top width 6 feet or as recommended by a civil engineer.

Water Supply

1) Water balance calculations should be provided to demonstrate that adequate water supply will be present to maintain a pool of water during a drought year when precipitation is 50% of average for the site. Water balance calculations should include evapotranspiration, infiltration, precipitation, spillway discharge, and dry weather flow (where appropriate).

2) Where water balance indicates that losses will exceed inputs, a source of water should be provided to maintain the basin water surface elevation throughout the year. The water supply should be of sufficient quantity and quality to not have an adverse impact on the wet detention basin water quality. Water that meets drinking water standards should be assumed to be of sufficient quality.

3) Wet detention basin may be designed as seasonal ponds where the water balance and water supply conditions make it infeasible to sustain a permanent wet detention basin.

Soils Considerations

Wet detention basin implementation in areas with high permeability soils requires liners to increase the chances of maintaining a permanent pool in the basin. Liners can be either synthetic materials or imported lower permeability soils (i.e., clays). The water balance assessment should determine whether a liner is required.

If low permeability soils are used for the liner, a minimum of 18 inches of native soil amended with good topsoil or compost (one part compost mixed with 3 parts native soil) should be placed over the liner. If a synthetic material is used, a soil depth of 2 feet is recommended to prevent damage to the liner during planting.

Buffer Zone

A minimum of 25 feet buffer should be provided around the top perimeter of the wet detention basin. The portion of the access road outside of the maximum water level may be included as part of the buffer.

Stormwater Quality Design Features

1) Wet detention basins that are located in publicly-accessible or highly visible locations should include design features that will improve and maintain the quality of water within the BMP at a level suitable for the proposed location and uses of the surrounding area. Typical design features include aeration, pumped circulation, filters, biofilters, and other facilities that operate year-round to remove pollutants and nutrients. Stormwater quality design features will result in higher quality water in the BMP and lower discharges of pollutants downstream.

2) Wet detention basins in publicly-accessible or highly visible locations should have a maintenance plan that includes regular collection and removal of trash from the area within and surrounding the BMP.

3) If fencing is required for wet detention basins in publicly-accessible or highly visible locations, the fence can be designed to be aesthetically incorporated into the site and Shrubs (approved, California-adapted species) can be used to hide the fencing. See vegetation section below.

Energy Dissipation

1) The inlet to the wet detention basin should be submerged with the inlet pipe invert a minimum of two feet from the basin bottom (not including sediment storage). The top of the inlet pipe should be submerged at least 1 foot, if possible. Intent: The inlet is submerged to dissipate energy of the incoming flow. The distance from the bottom is set to minimize resuspension of settled sediments. Alternative inlet designs that accomplish these objectives are acceptable.

2) Energy dissipation controls should also be used at the outlet from the wet detention basin unless the basin discharges to a stormwater conveyance system or hardened channel.

Vegetation

A plan should be prepared that indicates how aquatic, temporarily submerged areas (extended detention wet detention basins) and terrestrial areas will be stabilized with vegetation.

1) If the second cell of the wet detention basin is 3 feet or shallower, the bottom area should be planted with emergent wetland vegetation.

2) Emergent aquatic vegetation should be planted to cover 25-75% of the area of the permanent pool.

3) Outside of the basin, native vegetation adapted for site conditions should be used in non-irrigated sites.

4) The area surrounding a wet detention basin should be landscaped to minimize erosion and should adhere to the following criteria so as not to hinder maintenance operations:

5) No trees or shrubs may be planted within 15 feet of inlet or outlet pipes or manmade drainage structures such as spillways, flow spreaders, or earthen embankments. Species with roots that seek water, such as willow or poplar, should not be used within 50 feet of pipes or manmade structures. Weeping willow (Salix babylonica) should not be planted in or near detention basins.

6) Prohibited non-native plant species will not be permitted. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website- or the California Invasive Plant Council website at www.cal-ipc.org.

7) A landscape professional should provide recommendations on appropriate plants, fertilizer, mulching applications, and irrigation requirements (if any) to ensure healthy vegetation growth.

Outlet Structure

1) An outlet pipe and outlet structure should be provided. The outlet pipe may be a perforated standpipe strapped to a manhole or placed in an embankment, suitable for extended detention, or may be back-sloped to a catch basin with a grated opening (jail house window) or manhole with a cone grate (birdcage). The grate or birdcage openings provide an overflow route should the basin outlet pipe become clogged.

2) For extended detention wet detention basin, outlet structures should be designed to provide 12 to 48 hour emptying time for the water quality volume above the permanent pool.

3) The basin outlet pipe should be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for off-line basins or flows greater than the peak runoff discharge rate for the 100-year, 24-hr design storm for on-line basins.

Emergency Spillway

An emergency overflow spillway in addition to the primary overflow outlet (as described above) is required. The emergency spillway should be sized for flows greater than the peak 100-year 24-hour storm if the basin is designed on-line or, if the basin is designed off-line, the spillway should be sized for flows greater than the basin design volume (e.g., stormwater quality design volume). The spillway provide for adequate energy dissipation downstream. The spillway should allow for at least 12 inches of freeboard above the emergency overflow water surface elevation if the basin is on-line. If the basin is -line, 2 feet of freeboard is preferable.

Spillways shall meet the California Department of Water Resources, Division of Safety of Dams Guidelines for the Design and Construction of Small Embankment Dams (http://damsafety.water.ca.gov/docs/GuidelinesSmallDams.pdf). *Intent: Emergency overflow spillways are intended to control the location of basin overtopping and safely direct overflows back into the downstream conveyance system or other acceptable discharge point.*

On-line Basins

1) On-line basins must have an emergency overflow spillway to prevent overtopping of walls or berms should blockage of the primary outlet occur based on a downstream risk assessment.

2) The overflow spillway must be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm.

3) The minimum freeboard should be 1 foot (but preferably at least 2 feet) above the maximum water surface elevation over the emergency spillway.

Off-line Basins

1) Off-line basins must have either an emergency overflow spillway or an emergency overflow riser. The emergency overflow must be designed to pass flows greater than the basin design volume (e.g., stormwater quality design volume) directly to the downstream conveyance system or another acceptable discharge point. Where an emergency overflow spillway would discharge to a steep slope, an emergency overflow riser, in addition to the spillway should be provided. See Appendix E for basin/pond outlet sizing worksheets.

2) The emergency overflow spillway should be armored to withstand the energy of the spillway flows. The spillway should be constructed of grouted rip-rap.

3) The minimum freeboard should be 1 foot above the maximum water surface elevation over the emergency spillway.

Side Slopes

1) Interior side slopes above the stormwater quality design depth and up to the emergency overflow water surface steeper than 4:1 (H:V) should be stabilized to prevent erosion with a method approved by the local permitting authority.

2) Exterior side slopes steeper than 2:1 (H:V) should be stabilized to prevent erosion with a method approved by the local permitting authority.

3) For any slope (interior or exterior) greater than 2:1 (H:V), a geotechnical investigation and report must be submitted and approved by the local permitting authority.

4) Landscaped slopes should be no steeper than 3:1 (H:V) to allow for maintenance.

5) Basin walls may be vertical retaining walls, provided: (a) they are constructed of reinforced concrete, (b) a fence is provided along the top of the wall (see fencing below) or further back, and (c) the design is stamped by a licensed civil engineer.

Embankments

1) Earthworks and berm embankments should be performed in accordance with the latest edition of the "Greenbook Standard Specifications for Public Works Construction".

2) Embankments are earthen slopes or berms used for detaining or redirecting the flow of water.

3) Top of berm should be 2 feet minimum below the stormwater quality design water surface and should be keyed into embankment a minimum of 1 foot on both sides.

4) Typically, the top width of berm embankments are at least 20 feet, but narrower embankments may be plausible if approved by the civil engineer and the Local permitting authority.

5) Basin berm embankments should be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a licensed civil engineer) free of loose surface soil materials, roots, and other organic debris.

6) The berm embankment should be constructed of compacted soil (95% minimum dry density, modified proctor method per ASTM D1557), placed in 6-inch lifts.

7) Basin berm embankments greater than 4 feet in height should be constructed by excavating a key equal to 50% of the berm embankment cross-sectional height and width. This requirement may be waived if specifically recommended by a licensed civil engineer.

8) The berm embankment should be constructed of compacted soil (95% minimum dry density, modified proctor method per ASTM D1557), placed in 6-inch lifts.

9) Low growing native or non-invasive perennial grasses should be planted on downstream embankment slopes. See vegetation section below.

Fencing

Safety is provided either by fencing of the facility or by managing the contours of the basin to eliminate drop-offs and other hazards.

1) If fences are required, fences should be designed and constructed in accordance with current and relevant policies and typically are required to be located at or above the overflow water surface elevation. Shrubs (approved, California-adapted species) can be used to hide the fencing. See vegetation section above.

Right-of-Way

2) Wet detention basins and associated access roads to be maintained by a public agency should be dedicated in fee or in an easement to the public agency with appropriate access.

Maintenance Access

1) Ownership of the basin and maintenance thereof is the responsibility of the developer/applicant. A maintenance agreement is required to ensure adequate performance and allow emergency access to the facilities.

2) Maintenance access road(s) should be provided to the control structure and other drainage structures associated with the basin (e.g., inlet, emergency overflow or bypass structures). Manhole and catch basin lids should be in or at the edge of the access road.

3) A ramp into the basin should be constructed near the basin outlet. An access ramp is required for removal of sediment with a backhoe or loader and truck. The ramp should extend to the basin bottom to avoid damage to vegetation planted on the basin slope.

4) All access ramps and roads should be provided in accordance with the current policies of the Flood Control District.

Vector Control

1) A Mosquito Management Plan or Service Contract should be approved or waived by the local Vector Control District for any facility that maintains a pool of water for 72 hours or more.

Operations and Maintenance

General Requirements

Maintenance is of primary importance if extended detention basins are to continue to function as originally designed. A maintenance agreement must be developed with the Flood Control District to ensure adequate performance and allow the County emergency access. Maintenance of the basin is the responsibility of the development, unless otherwise agreed upon.

A specific maintenance plan shall be formulated for each facility outlining the schedule and scope of maintenance operations, as well as the data handling and reporting requirements. The following are general maintenance requirements:

1) The basin should be inspected annually and inspections after major storm events are encouraged (see Appendix I for guidance on facility maintenance inspections). Trash and debris should be removed as needed, but at least annually prior to the beginning of the wet season (see Appendix I for dry extended detention basin inspection and maintenance checklist).

2) Site vegetation should be maintained as follows:

3) Vegetation, large shrubs, or trees that limit access or interfere with basin operation should be pruned or removed.

4) Slope areas that have become bare should be revegetated and eroded areas should be regraded prior to being revegetated.

5) Grass should be mowed to 4"-9" high and grass clippings should be removed.

6) Fallen leaves and debris from deciduous plant foliage should be raked and removed.

7) Invasive vegetation, such as Alligatorweed (Alternanthera philoxeroides), Halogeton (Halogeton glomeratus), Spotted Knapweed (Centaurea maculosa), Giant Reed (Arundo donax), Castor Bean (Ricinus communis), Perennial Pepperweed (Lepidium latifolium), and Yellow Starthistle (Centaurea solstitalis) should be removed and replaced with non-invasive species. Invasive species should never contribute more than 25% of the vegetated area. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website or the California Invasive Plant Council website at www.cal-ipc.org.

8) Dead vegetation should be removed if it exceeds 10% of area coverage. Vegetation should be replaced immediately to maintain cover density and control erosion where soils are exposed.

9) No herbicides or other chemicals should be used to control vegetation.

10) Sediment buildup exceeding 50% of the forebay capacity should be removed. Sediment from the remainder of the basin should be removed when 6 inches of sediment accumulates. Sediments should be tested for toxic substance accumulation in compliance with current disposal requirements if land uses in the catchment include commercial or industrial zones, or if visual or olfactory indications of pollution are noticed. If toxic substances are encountered at concentrations exceeding thresholds of Title 22, Section 66261 of the California Code of Regulations, the sediment must be disposed of in a hazardous waste landfill.

11) Following sediment removal activities, replanting, and/or reseeding of vegetation may be required for reestablishment.

Construction Considerations

The use of treated wood or galvanized metal anywhere inside the facility is prohibited. The use of galvanized fencing is permitted if in accordance with the Fencing requirement above.

TCM-3: Constructed Wetland

A constructed treatment wetland is a system consisting of a sediment forebay and one or more permanent micro-pools with aquatic vegetation covering a significant portion of the basin. Constructed treatment wetlands typically include components such as an inlet with energy dissipation, a sediment forebay for settling out coarse solids and to facilitate maintenance, a base with shallow sections (1 to 2 feet deep) planted with emergent vegetation, deeper areas or micro pools (3 to 5 feet deep), and a water quality outlet structure. The interactions between the incoming stormwater runoff, aquatic vegetation, wetland soils, and the associated physical, chemical, and biological unit processes are a fundamental part of constructed treatment wetlands.

Constructed Wetlands

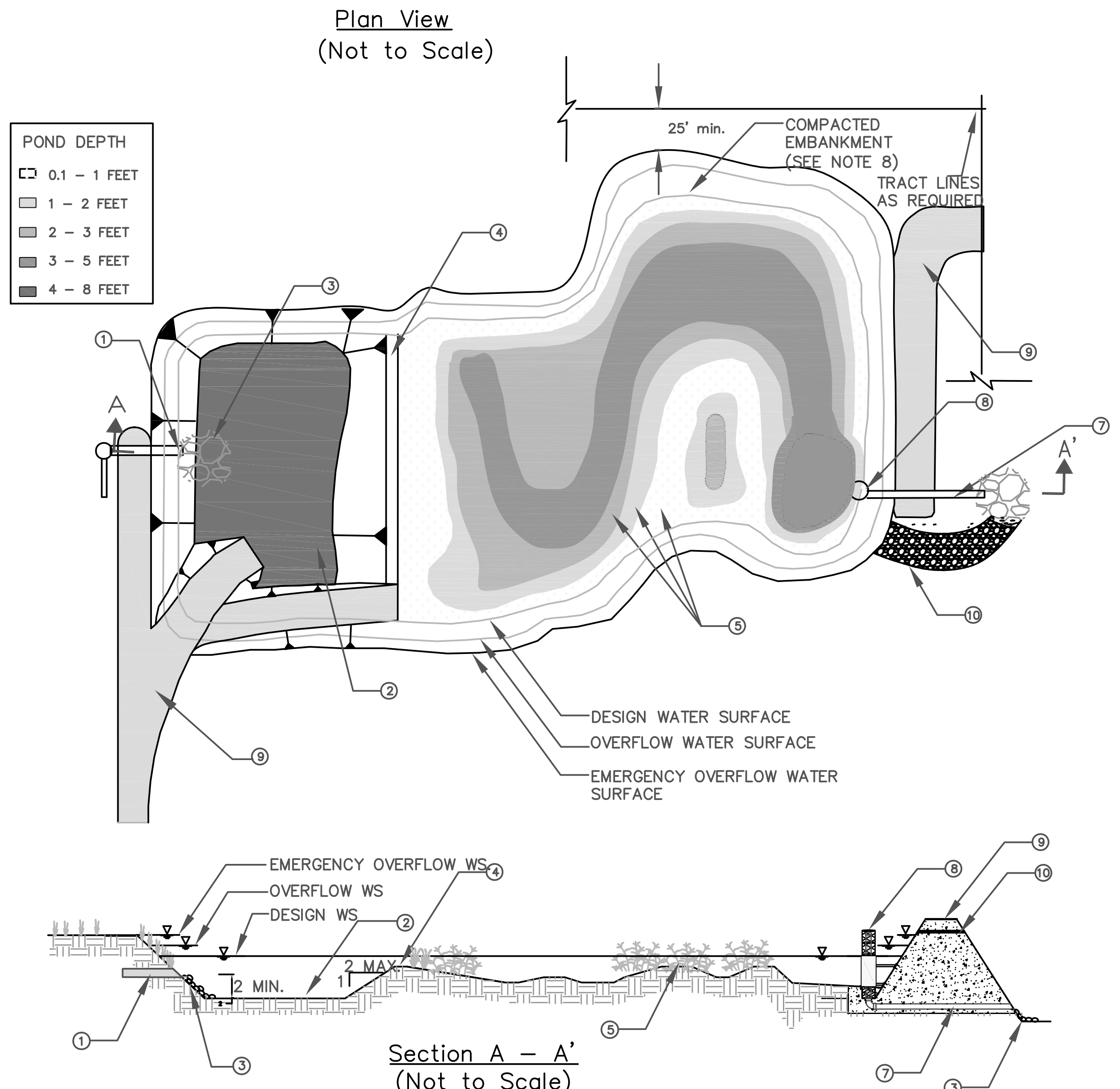
Photo Credits: Geosyntec Consultants

Application

- Regional detention & treatment
- Roads, highways, parking lots, commercial, residential
- Parks, open spaces, and golf courses

Preventative Maintenance

- inspected at a minimum annually and inspections after major storm events
- Pruned or remove vegetation, large shrubs, or trees that limit access or interfere with basin operation
- Remove sediment buildup at inlets and outlets



NOTES:

1. INLET PIPE SHOULD BE SUBMERGED WITH A MINIMUM OF 2' DISTANCE FROM THE BOTTOM
2. SEDIMENT FOREBAY. FORE BAY VOLUME SHALL EQUAL 10% TO 20% OF TOTAL WETLAND VOLUME. FOREBAY DEPTH SHALL BE 4' MIN TO 8' MAX PLUS AN ADDITIONAL I' MIN SEDIMENT STORAGE DEPTH.
3. RIP RAP APRON OR OTHER INLET ENERGY DISSIPATION.
4. BERM AT DESIGN WATER SURFACE ELEVATION OR SUBMERGED 1' BELOW DESIGN WATER SURFACE ELEVAVTION. EXTEND BERM ACROSS ENTIRE WIDTH OF THE WETLAND.
5. WETLAND VEGETATION. PLANTING SCHEME MUST BE DESIGNED BY A WETLAND ECOLOGIST.
6. EMBANKMENT SIDE SLOPES SHALL BE NO STEEPER THAN 2H:1V OUTSIDE AND 3H:1V INSIDE
7. SIZE OUTLET PIPE TO PASS 100-YEAR PEAK FLOW FOR ONLINE AND WATER QUALTIY PEAK FLOW FOR OFFLINE BASINS.
8. WATER QUALITY OUTLET STRUCTURE. SEE FIGURE 7-2 AND FIGURE 7-3 FOR DETAILS.
9. MAINTENANCE RAMP SHOULD PROVIDE ACCESS TO BOTH THE FOREBAY AND MAIN BASIN.
10. INSTALL EMERGENGY OVERFLOW SPILWAY AS NEEDED. SEE FIGURE 2-4 FOR DETAILS

Geosyntec consultants

Figure 6-22: Constructed Wetland

Limitations

- In theory, there are no limitations on the tributary area size draining to a constructed treatment wetland; however, constructed treatment wetlands usually require considerable land area. Typically, treatment wetlands capture runoff from tributary areas larger than 10 acres and less than 10 square miles. Smaller "pocket" wetlands can be feasible in areas where space is restricted.
- If the constructed treatment wetland is not used for flow control, the wetland must not interfere with flood control functions of existing conveyance and detention structures.
- Constructed treatment wetlands should not be permitted in areas with site slopes greater than 7% or within 200 feet (on the uphill side) of a steep slope hazard area or a mapped landslide area unless a geotechnical investigation and report is completed by a licensed civil engineer.
- Constructed treatment wetlands require a regular source of water (base flow) to maintain wetland vegetation and associated treatment processes. If adequate base flow is not available year-round, supplemental water may be needed during the summer months to maintain adequate base flow.

Design Criteria

The main challenge associated with constructed treatment wetlands is maintaining base flow to support vegetation. Constructed wetlands should be designed according to the requirements listed in Table 6-25 and outlined in the section below. Constructed wetland BMP sizing worksheets are presented in Appendix E.

Table 6-25: Constructed Wetland Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume, SQDV	acre-feet	See Section 2 and Appendix E for calculating SQDV.
Permanent pool volume	%	75% of SQDV
Drawdown time for extended detention (over permanent pool)	hours	48 ; 12 for 50% SQDV (minimum)
Sediment forebay volume	%	30 to 50% of permanent pool surface area
Depth of sediment forebay	feet	2-4 (1 foot of sediment storage required)
Wetland zone volume	%	50-70% of permanent pool surface area
Depth of wetland basin	feet	0.5 to 1.0 (30 to 50% should be 0.5 feet deep)

Design Parameter	Unit	Design Criteria
Wetland (littoral zone) bottom slope	%	10 maximum
Maximum residence time	Days	7 (dry weather)
Freeboard (minimum)	inches	12
Flow path length to width ratio	L:W	2:1, larger preferred
Side slope (maximum)	H:V	4:1 Interior; 3:1 Exterior
Vegetation Type	--	Varies see vegetation section below
Vegetation Height	--	Varies see vegetation section below
Buffer zone (minimum)	feet	25
Minimum outflow device diameter	inches	18

Sizing

In most cases, the constructed treatment wetland permanent pool should be sized to be greater than or equal to the stormwater quality design volume. If extended detention is provided above the permanent pool and the wetland is designed for water quality treatment only, then the permanent pool volume should be a minimum of 80 percent of the stormwater quality design volume and the surcharge volume (above the permanent pool) should make up the remaining 20 percent and provide at least 12 hours of detention. If extended detention is provided and the basin is designed for water quality treatment and peak flow attenuation, then the permanent pool volume should be equal to the water quality treatment volume and the surcharge volume should be sized to attenuate peak flows to meet the peak runoff discharge requirements. The extended detention portion of the wetland above the permanent pool, if provided, functions like a dry extended detention (ED) basin (see VEG-5: Dry Extended Detention Basin).

Step 1: Calculate the design volume

Constructed wetlands shall be sized to be greater than or equal to the SQDV volume (see Section 2 and Appendix E).

Step 2: Determine the Wetland Location, Wetland Type and Preliminary Geometry Based on Site Constraints

Based on site constraints, determine the wetland geometry and the storage available by developing an elevation-storage relationship for the wetland. The equations provided below assume a trapezoidal geometry for cell 1 (Forebay) and cell 2, and assumes that the wetland does not have extended detention.

1) Calculate the width of the wetland footprint, W_{tot}, as follows:

$$W_{tot} = \frac{A_{tot}}{L_{tot}} \qquad \text{(Equation 6-81)}$$

Where:

A_{tot}	=	total surface area of the wetland footprint (ft^2)
L_{tot}	=	total length of the wetland footprint (ft)

2) Calculate the length of the water quality volume surface area including the internal berm but excluding the freeboard, $L_{wq\text{-}tot}$:

$$L_{wq-tot} = L_{tot} - 2Zd_{fb} \qquad \text{(Equation 6-82)}$$

Where:

Z	=	interior side slope as length per unit height
d_{fb}	=	freeboard depth

3) Calculate the width of the water quality volume surface area including the internal berm but excluding freeboard, $W_{wq\text{-}tot}$:

$$W_{wq-tot} = W_{tot} - 2Zd_{fb} \qquad \text{(Equation 6-83)}$$

4) Calculate the total water quality volume surface area including the internal berm and excluding freeboard, $A_{wq\text{-}tot}$:

$$A_{wq-tot} = L_{wq-tot} \times W_{wq-tot} \qquad \text{(Equation 6-84)}$$

5) Calculate the area of the berm, A_{berm}:

$$A_{berm} = W_{berm} \times L_{berm} \qquad \text{(Equation 6-85)}$$

Where:

W_{berm}	=	width of the internal berm
L_{berm}	=	length of the internal berm

6) Calculate the water quality surface area excluding the internal berm and freeboard, A_{wq}:

$$A_{wq} = A_{wq\,=\,tot} - A_{berm}$$ (Equation 6-86)

Step 3: Determine Dimensions of Forebay

30-50% of the SQDV is required to be within the active volume of forebay.

1) Calculate the active volume of forebay, V_1:

$$V_1 = \frac{SQDV \times \%V_1}{100}$$ (Equation 6-87)

Where:

$\%V_1$ = percent of SQDV in forebay (%)

2) Calculate the surface area for the active volume of forebay, A_1:

$$A_1 = \frac{V_1}{d_1}$$ (Equation 6-88)

Where:

d_1 = average depth fo rhte active volume of forebay (2 -4 ft) (ft)

3) Calculate the length of forebay, L_1. Note, inlet and outlet should be configured to maximize the residence time.

$$L_1 = \frac{A_1}{W_1}$$ (Equation 6-89)

Where:

W_1 = width of forebay (ft), $W_1 = W_{av\text{-}tot} = L_{berm}$

Step 4: Determine Dimensions of Cell 2

Cell 2 will consist of the remainder of the basin's active volume.

1) Calculate the active volume of Cell 2, V_2:

$$V_2 = SQDV - V_1$$ (Equation 6-90)

2) Calculate the surface area of Cell 2, A_2:

$$A_2 = A_{wq} - A_1$$ (Equation 6-91)

3) Calculate the top length of Cell 2, L_2:

$$L_2 = \frac{A_2}{W_2} \qquad \text{(Equation 6-92)}$$

Where:

W_2 = width of Cell 2 (ft), $W_2 = W_1 = W_{wq\text{-}tot} = L_{berm}$

4) Verify that the length-to-width ratio of Cell 2, LW_2, is at least 3:1 with ≥ 4:1 preferred. If the length-to-width ratio is less than 3:1, modify input parameters until a ratio of at least 3:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the pond should be chosen.

$$LW_2 = \frac{L_2}{W_2} \qquad \text{(Equation 6-93)}$$

5) Calculate the very shallow zone surface area, A_{vs}:

$$A_{vs} = \frac{A_2 \bullet \%A_{vs}}{100} \qquad \text{(Equation 6-94)}$$

Where:

$\%A_{vs}$ = percent of surface area of very shallow zone

6) Calculate the volume of the shallow zone, V_{vs}:

$$V_{vs} = A_{vs} \bullet d_{vs} \qquad \text{(Equation 6-95)}$$

Where:

d_{vs} = average depth of the very shallow zone (0.1 – 1 ft)

7) Calculate the length of the very shallow zone, L_{vs}:

$$L_{vs} = \frac{A_{vs}}{W_{vs}} \qquad \text{(Equation 6-96)}$$

Where:

W_{vs} = width of the very shallow zone (ft), $W_{vs} = W_2$

8) Calculate the surface area of the shallow zone, A_s:

$$A_s = \frac{A_2 \bullet \%A_s}{100} \qquad \text{(Equation 6-97)}$$

Where:

$\%A_s$ = percent of surface area of shallow zone

9) Calculate the volume of the shallow zone, V_s:

$$V_s = A_s \bullet d_s \qquad \text{(Equation 6-98)}$$

Where:

d_s = average depth of shallow zone (1 - 3 ft)

10) Calculate length of the shallow zone, L_s:

$$L_s = \frac{A_s}{W_s} \qquad \text{(Equation 6-99)}$$

Where:

W_s = width of the shallow zone (ft), $W_s = W_2$

11) Calculate the surface area of the deep zone, A_{deep}:

$$A_{deep} = A_2 - A_{vs} - A_s \qquad \text{(Equation 6-100)}$$

12) Calculate the volume of the deep zone, V_{deep}:

$$V_{deep} = V_2 - V_{vs} - V_s \qquad \text{(Equation 6-101)}$$

13) Calculate the average depth of the deep zone (3-5 ft), d_{deep}:

$$d_{deep} = \frac{V_{deep}}{A_{deep}} \qquad \text{(Equation 6-102)}$$

14) Calculate length of the deep zone, L_{deep}:

$$L_{deep} = \frac{A_{deep}}{W_{deep}} \qquad \text{(Equation 6-103)}$$

Where:

W_{deep} = width of the deep zone (ft), $W_{deep} = W_2$

Step 5: Ensure design requirements and site constraints are achieved

Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location or alternative treatment BMP.

Step 6: Size Outlet Structure

For wetlands with detention, the outlet structures should be designed to provide 12 hours emptying time for the water quality volume or the required detention necessary for achieving the peak runoff discharge requirements if the extended detention is designed for flow attenuation.

The wetland outlet pipe should be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for on-line basins or flows greater than the peak runoff discharge rate for the 100-year, 24-hr design storm for on-line basins.

Step 7: Determine Emergency Spillway Requirements

For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm in order to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the 100-yr, 24-hr post-development peak storm water runoff discharge rate directly to the downstream conveyance system or another acceptable discharge point. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

Sizing and Geometry

In most cases, the constructed treatment wetland permanent pool should be sized to be greater than or equal to the stormwater quality design volume. If extended detention is provided above the permanent pool and the wetland is designed for water quality treatment only, then the permanent pool volume should be a minimum of 80 percent of the stormwater quality design volume and the surcharge volume (above the permanent pool) should make up the remaining 20 percent and provide at least 12 hours of detention. If extended detention is provided and the basin is designed for water quality treatment and peak flow attenuation, then the permanent pool volume should be equal to the water quality treatment volume and the surcharge volume should be sized to attenuate peak flows to meet the peak runoff discharge requirements. A constructed treatment wetland design worksheets are presented in Appendix E. The extended detention portion of the wetland above the permanent pool, if provided, functions like a dry extended detention (ED) basin (see TCM-1: Dry Extended Detention Basin).

1) Constructed treatment wetlands should consist of at least two cells including a sediment forebay and a wetland basin.

2) The sediment forebay must contain between 10 and 20 percent of the total basin volume.

3) The depth of the sediment forebay should be between 4 and 8 feet.

4) One foot of sediment storage should be provided in the sediment forebay.

5) The “berm” separating the two basins should be uniform in cross-section and shaped such that its downstream side gradually slopes to the main wetland basin.

6) The top of berm should be either at the stormwater quality design water surface or submerged 1 foot below the stormwater quality design water surface, as with wet retention basins. Correspondingly, the side slopes of the berm should meet the following criteria:

    a. If the type of the berm is at the stormwater quality design water surface, the berm side slopes should be no steeper than 4H:1V.

    b. If the top of berm is submerged 1 foot, the upstream side slope may be a max of 3H:1V.

7) The constructed treatment wetlands should be designed with a “naturalistic” shape and a range of depths intermixed throughout the wetland basin to a maximum of 5 feet.

Depth Range (feet)	Percent by Area
0.1 to 1	15
1 to 3	55
3 to 5	30

8) The flowpath length-to-width ratio should be a minimum of 2:1, but preferably at least 4:1 or greater. *Intent: a high flow path length to width ratio will maximize fine sediment removal.*

9) The minimum freeboard should be 1 foot above the maximum water surface elevation for on-line basins (2 feet preferable) and 1 foot above the maximum water surface elevation for on-line basins.

10) Wetland pools should be designed such that the residence time for dry weather flows is no greater than 7 days. *Intent: Minimize vector and stagnation issues.*

Water Supply

Water balance calculations should be provided to demonstrate that adequate water supply will be present to maintain a permanent pool of water during a drought year when precipitation is 50% of average for the site. Water balance calculations should include evapotranspiration, infiltration, precipitation, spillway discharge, and dry weather flow (where appropriate).

Where water balance indicates that losses will exceed inputs, a source of water should be provided to maintain the wetland water surface elevation throughout the year. The water supply should be of sufficient quantity and quality to not have an adverse impact on the wetland water quality. Water that meets drinking water standards should be assumed to be of sufficient quality.

Soils Considerations

1) Implementation of constructed treatment wetlands in areas with high permeability soils (>0.1 in/hr) requires liners to increase the chances of maintaining permanent pools and/or micro-pools in the basin. Liners can be either synthetic materials or imported lower permeability soils (i.e., clays). The water balance assessment should determine whether a liner is required. The following conditions can be used as a guideline.

2) The wetland basin should retain water for at least 10 months of the year.

3) The sediment forebay should retain at least 3 feet of water year-round.

4) Many wetland plants can adapt to periods of summer drought, so a limited drought period is allowed in the wetland basin. This may allow for a soil liner rather than a geosynthetic liner. The sediment forebay should retain water year-round for presettling to be effective.

5) If low permeability soils are used for the liner, a minimum of 18 inches of native soil amended with good topsoil or compost (one part compost mixed with 3 parts native soil) should be placed over the liner (see soil amendment Section 5.10). If a synthetic material is used, a soil depth of 2 feet is recommended to prevent damage to the liner during planting.

Buffer Zone

A minimum of 25 feet buffer should be provided around the top perimeter of the constructed treatment wetlands.

Energy Dissipation

1) The inlet to the constructed treatment wetland should be submerged with the inlet pipe invert a minimum of two feet from the cell bottom (not including sediment storage). The top of the inlet pipe should be submerged at least 1 foot, if possible. *Intent: the inlet is submerged to dissipate energy of the incoming flow. The distance from the bottom is set to minimize resuspension of settled sediments. Alternative inlet designs that accomplish these objectives are acceptable.*

2) Energy dissipation controls must also be used at the outlet/spillway from the constructed treatment wetlands unless the wetland discharges to a stormwater conveyance system or hardened channel.

Vegetation

1) The wetland cell(s) should be planted with emergent wetland plants following the recommendations of a wetlands specialist.

2) Landscaping outside of the basin is required for all constructed wetlands and should adhere to the following criteria so as not to hinder maintenance operations:

    a. No trees or shrubs may be planted within 15 feet of inlet or outlet pipes or manmade drainage structures such as spillways, flow spreaders, or earthen embankments. Species with roots that seek water, such as willow or poplar, should not be used within 50 feet of pipes or manmade structures. Weeping willow (*Salix babylonica*) should not be planted in or near detention basins.

    b. Prohibited non-native plant species will not be permitted. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website or the California Invasive Plant Council website at www.cal-ipc.org.

3) Project-specific planting recommendations should be provided by a wetland ecologist or a qualified landscape professional including recommendations on appropriate plants, fertilizer, mulching applications, and irrigation requirements (if any) to ensure healthy vegetation growth.

Outlet Structure

An outlet pipe and outlet structure should be provided. The outlet pipe may be a perforated standpipe strapped to a manhole or placed in an embankment, suitable for extended detention, or may be back-sloped to a catch basin with a grated opening (jail house window) or manhole with a cone grate (birdcage). The grate or birdcage openings provide an overflow route should the basin outlet pipe become clogged. The outlet should be protected from clogging by a skimmer shield that starts at the bottom of the permanent pool and extends above the SQDV depth. A trash rack is also required.

For wetlands with detention, the outlet structures should be designed to provide 12 hours emptying time for the water quality volume or the required detention necessary for achieving the peak runoff discharge requirements if the extended detention is designed for flow attenuation.

The wetland outlet pipe should be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for on-line basins or flows greater than the peak runoff discharge rate for the 100-year, 24-hr design storm for on-line basins.

See the dry extended detention section (see ST-1: Dry Extended Detention Basin) and Appendix E for further detail on outlet sizing.

Emergency Spillway

An emergency overflow spillway in addition to the primary overflow outlet (as described above) is required. The emergency spillway should be sized for flows greater than the peak 100-year 24-hour storm if the basin is designed on-line or, if the basin is designed on-line, the spillway should be sized for flows greater than the basin design volume (e.g., stormwater quality design volume). The spillway provide for adequate energy dissipation downstream. The spillway should allow for at least 12 inches of freeboard above the emergency overflow water surface elevation if the basin is on-line. If the basin is on-line, 2 feet of freeboard is preferable.

Spillways shall meet the California Department of Water Resources, Division of Safety of Dams Guidelines for the Design and Construction of Small Embankment Dams (http://damsafety.water.ca.gov/docs/GuidelinesSmallDams.pdf). *Intent: Emergency overflow spillways are intended to control the location of basin overtopping and safely direct overflows back into the downstream conveyance system or other acceptable discharge point.*

On-line Basins

1) On-line basins must have an emergency overflow spillway to prevent overtopping of walls or berms should blockage of the primary outlet occur based on a downstream risk assessment.

2) The overflow spillway must be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm.

3) The minimum freeboard should be 1 foot (but preferably at least 2 feet) above the maximum water surface elevation over the emergency spillway.

Off-line Basins

1) Off-line basins must have either an emergency overflow spillway or an emergency overflow riser. The emergency overflow must be designed to pass the 100-yr 24-hr post-development peak stormwater runoff discharge rate (see Appendix E for further detail) directly to the downstream conveyance system or another acceptable discharge point. Where an emergency overflow spillway would discharge to a steep slope, an emergency overflow riser, *in addition* to the spillway should be provided.

2) The emergency overflow spillway should be armored to withstand the energy of the spillway flows. The spillway should be constructed of grouted rip-rap.

3) The minimum freeboard should be 1 foot above the maximum water surface elevation over the emergency spillway.

Side Slopes

1) Interior side slopes above the stormwater quality design depth and up to the emergency overflow water surface steeper than 4:1 (H:V) should be stabilized to prevent erosion with a method approved by the local permitting authority.

2) Exterior side slopes steeper than 2:1 (H:V) should be stabilized to prevent erosion with a method approved by the local permitting authority.

3) For any slope (interior or exterior) greater than 2:1 (H:V), a geotechnical investigation and report must be submitted and approved by the local permitting authority.

4) Landscaped slopes should be no steeper than 3:1 (H:V) to allow for maintenance.

5) Basin walls may be vertical retaining walls, provided: (a) they are constructed of reinforced concrete, (b) a fence is provided along the top of the wall (see fencing below) or further back, and (c) the design is stamped by a licensed civil engineer and approved by the local permitting authority.

Embankments

1) Earthworks and berm embankments should be performed in accordance with the latest edition of the "Greenbook Standard Specifications for Public Works Construction".

2) Embankments are earthen slopes or berms used for detaining or redirecting the flow of water.

3) Top of berm should be 2 feet minimum below the stormwater quality design water surface and should be keyed into embankment a minimum of 1 foot on both sides.

4) Typically, the top width of berm embankments are at least 20 feet, but narrower embankments may be plausible if approved by the civil engineer and the local permitting authority.

5) Basin berm embankments should be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a licensed civil engineer) free of loose surface soil materials, roots, and other organic debris.

6) Basin berm embankments greater than 4 feet in height should be constructed by excavating a key equal to 50% of the berm embankment cross-sectional height and width. This requirement may be waived if specifically recommended by a licensed civil engineer.

7) The berm embankment should be constructed of compacted soil (95% minimum dry density, modified proctor method per ASTM D1557), placed in 6-inch lifts.

8) Low growing native or non-invasive perennial grasses should be planted on downstream embankment slopes. See vegetation section below.

Fencing

Safety is provided either by fencing of the facility or by managing the contours of the basin to eliminate drop-offs and other hazards.

1) Provide fencing in accordance with the local permitting agency's requirements Perimeter fencing (minimum height of 42 inches) should be required on all basins exceeding two feet in depth or where interior side slopes are steeper than 6:1 (H:V).

2) If fences are required, fences should be designed and constructed in accordance with current policies of the local permitting agency and should be located at or above the overflow water surface elevation. Shrubs (approved, California-adapted species) can be used to hide the fencing. See vegetation section above.

Right-of-Way

1) Constructed treatment wetlands and associated access roads to be maintained by a public agency should be dedicated in fee or in an easement to the public agency with appropriate access.

Maintenance Access

1) Ownership of the basin and maintenance thereof is the responsibility of the developer/applicant. A maintenance agreement is required to ensure adequate performance and allow emergency access to the facilities.

2) Maintenance access road(s) should be provided to the control structure and other drainage structures associated with the basin (e.g., inlet, emergency overflow or bypass structures). Manhole and catch basin lids should be in or at the edge of the access road.

3) An access ramp into the basin should be constructed near the basin outlet. An access ramp is required for removal of sediment with a backhoe or loader and truck. The ramp should extend to the basin bottom to avoid damage to vegetation planted on the basin slope.

4) All access ramps and roads should be provided in accordance with the current policies of the Flood Control District.

Vector Control

1) A Mosquito Management Plan or Service Contract should be approved or waived by the local Vector Control District for any facility that maintains a pool of water for 72 hours or more.

Construction Considerations

The use of treated wood or galvanized metal anywhere inside the facility is prohibited. The use of galvanized fencing is permitted if in accordance with the Fencing requirement above.

Operations and Maintenance

Maintenance is of primary importance if constructed treatment wetlands basins are to continue to function as originally designed. A specific maintenance plan shall be formulated for each facility outlining the schedule and scope of maintenance operations, as well as the data handling and reporting requirements. The following are general maintenance requirements:

1) The constructed treatment wetlands basin should be inspected twice annually or more frequently, and inspections after major storm events are encouraged (see Appendix I for a constructed treatment wetland inspection and maintenance checklist). Trash and debris should be removed as needed, but at least annually prior to the beginning of the wet season.

2) Site vegetation should be maintained as frequently as necessary to maintain the aesthetic appearance of the site and to prevent clogging of outlets, creation of dead volumes, and barriers to mosquito fish to access pooled areas, and as follows:

3) Vegetation, large shrubs, or trees that limit access or interfere with basin operation should be pruned or removed.

4) Slope areas that have become bare should be revegetated and eroded areas should be regraded prior to being revegetated.

5) Invasive vegetation, such as Alligatorweed (*Alternanthera philoxeroides*), Halogeton (*Halogeton* glomeratus), Spotted Knapweed (*Centaurea maculosa*), Giant Reed (*Arundo donax*), Castor Bean (*Ricinus communis*), Perennial Pepperweed (*Lepidium latifolium*), and Yellow Starthistle (*Centaurea solstitalis*) should be removed and replaced with non-invasive species. Invasive species should never contribute more than 25% of the vegetated area. For more information on invasive weeds, including biology and control of listed weeds, look at the encycloweedia located at the California Department of Food and Agriculture website or the California Invasive Plant Council website at www.cal-ipc.org.

6) Dead vegetation should be removed if it exceeds 10% of area coverage. This does not include seasonal die-back where roots would grow back later in colder areas. Vegetation should be replaced immediately to maintain cover density and control erosion where soils are exposed.

7) Sediment buildup exceeding 6 inches over the storage capacity in the first cell should be removed. Sediments should be tested for toxic substance accumulation in compliance with current disposal requirements if land uses in the catchment include

commercial or industrial zones, or if visual or olfactory indications of pollution are noticed. If toxic substances are encountered at concentrations exceeding thresholds of Title 22, Section 66261 of the California Code of Regulations, the sediment must be disposed of in a hazardous waste landfill. Clean forebay every two years at a minimum, to avoid accumulation in main wetland area. Environmental regulations and permits may be involved with the removal of wetland deposits. When the main wetland area needs to be cleaned, it is suggested that the main area be cleaned one half at a time with at least one growing season in between cleanings. This will help to preserve the vegetation and enable the wetland to recover more quickly from the cleaning.

8) Repair erosion to banks and bottom as required.

9) Inspect outlet for clogging a minimum of twice a year, before and after the rainy season, after large storms, and more frequently if needed. Correct observed problems as necessary.

10) Following sediment removal activities, replanting, and/or reseeding of vegetation may be required for reestablishment.

TCM-4: Sand Filters

Sand filters operate much like bioretention facilities; however, instead of filtering stormwater through engineered soils, stormwater is filtered through a constructed sand bed with an underdrain system. Runoff enters the filter and spreads over the surface. As flows increase, water backs up on the surface of the filter where it is held until it can percolate through the sand. The treatment pathway is vertical (downward through the sand) to a perforated underdrain system that is connected to the downstream storm drainage system or to an infiltration facility. As stormwater passes through the sand, pollutants are trapped in the small pore spaces between sand grains or are adsorbed to the sand surface.

Sand filters connected to impervious surfaces

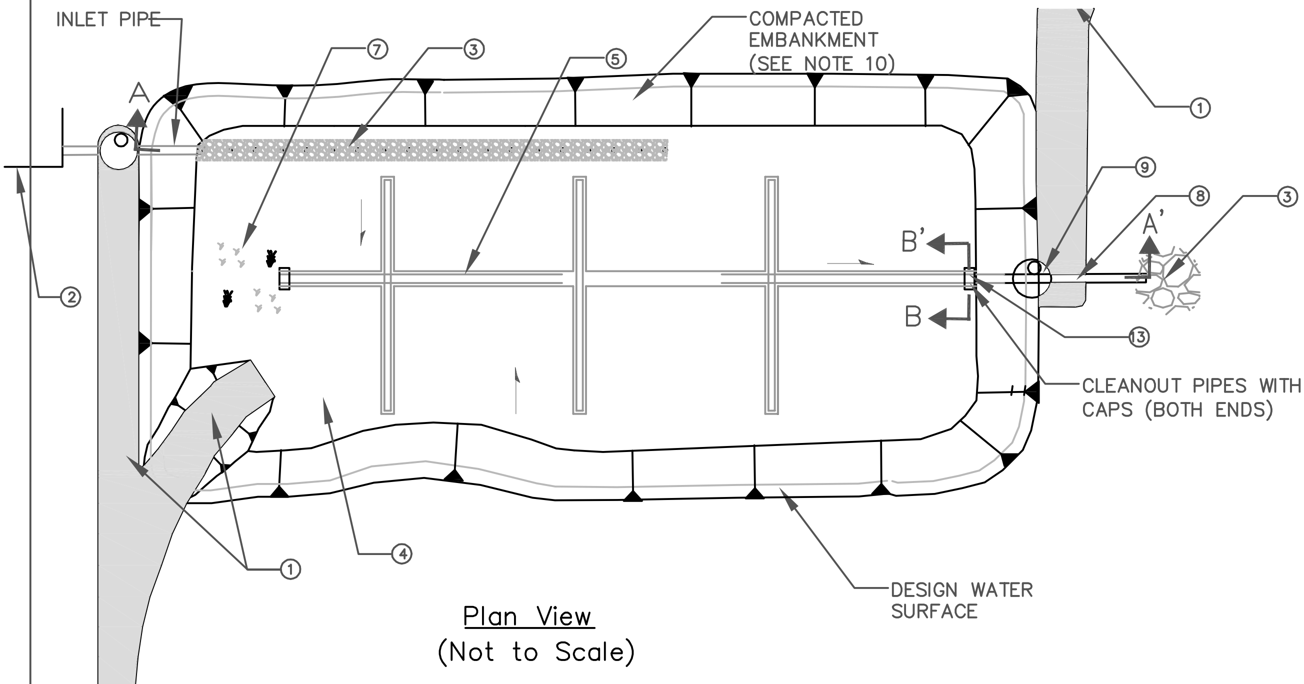
Photo Credits: Geosyntec Consultants

Application

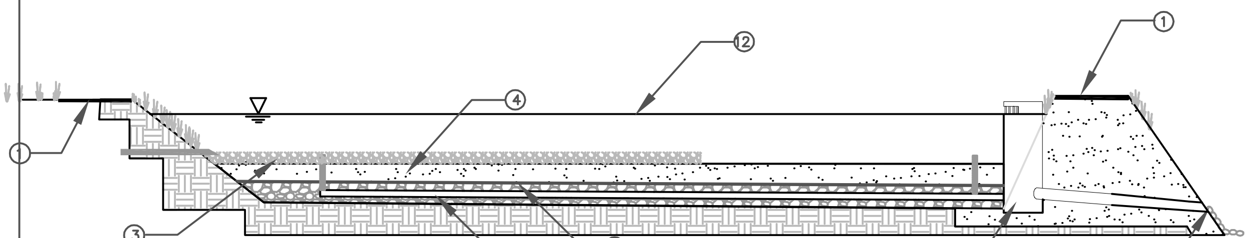
- Adjacent to parking lots
- Road medians and shoulders
- Within open areas or play fields

Preventative Maintenance

- Remove trash and debris, minor sediment accumulation, and obstructions near inlet and outlet structures
- Replace top 2” – 4” of sand
- Mow or weed surface of filter



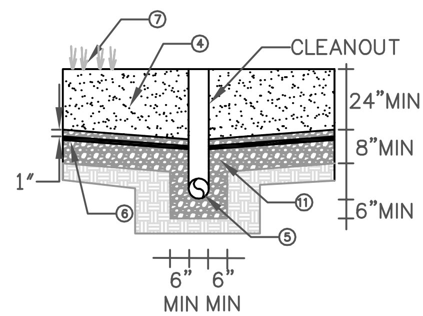
Plan View
(Not to Scale)



Section A – A'
(Not to Scale)

NOTES:

1. INSTALL MAINTENANCE ACCESS ROAD AND RAMP TO BOTTOM OF SAND FILTER.
2. UPSTREAM PRETREATMENT SHALL BE PROVIDED. IN THE ABSENCE OF PRETREATMENT, INCLUDE SEDIMENT FOREBAY WITH VOLUME EQUAL TO 10–20% OF TOTAL SAND FILTER VOLUME.
3. FLOW SPREADER TO EVENLY DISTRIBUTE FLOWS ALONG AT LEAST 20% OF PERIMETER.
4. FILTER BED SHALL BE A 24" MINIMUM SAND LAYER ON TOP OF 8" MINIMUM GRAVEL OR DRAIN ROCK BACKFILL.
5. 6" MINIMUM DIAMETER PERFORATED PIPE UNDERDRAIN SURROUNDED BY GRAVEL BEDDING. INSTALL AT 0.5% MINIMUM SLOPE
6. INSTALL GEOTEXTILE FABRIC OVERLAIN BY 1" OF DRAIN ROCK OR TRANSITIONALLY GRADED AGGREGRATE BETWEEN SAND AND GRAVEL LAYER.
7. VEGETATION MAY BE PLANTED ON TOP OF FILTER BED. NO TOP SOIL SHALL BE ADDED TO FILTER BED.
8. SIZE OUTLET PIPE STRUCTURE TO PASS WATER QUALITY DESIGN STORM AND INCLUDE AN EMERGENCY OVERFLOW.
9. EMERGENCY OVERFLOW STRUCTURE.
10. ¾" – 1½" WASHED DRAIN ROCK OR GRAVEL LAYER.
11. DESIGN WATER SURFACE. 6' MAX PONDING DEPTH.



Section B – B'
(Not to Scale)

Geosyntec consultants

Figure 6-23: Sand Filter

Limitations

Limitations for sand filters include:

- The sand filter should be located away from trees producing leaf litter or areas contributing significant eroded sediment to prevent clogging.
- Sand filters are should not be used in areas where heavy sediment loads are expected or in tributary areas that are not fully stabilized; high sediment loading rates may cause premature clogging of the filter. Pretreatment is essential.
- Site must have adequate relief between land surface and stormwater conveyance system to permit vertical percolation through the sand filter and collection and conveyance in the underdrain to stormwater conveyance system; four feet of elevation difference is recommended between the inlet and outlet of the filter.
- Not applicable in areas of high groundwater.
- Does not provide quantity control.

Design Criteria

The main challenge associated with sand filters is maintaining the filtration capacity, which is critical to the performance of this BMP. If flows entering the sand filter have high sediment concentrations, clogging of the sand filter is likely. Contribution of eroded soils or leaf litter may also reduce the infiltration and associated treatment capacity of the structure. Sand filters should be designed according to the requirements listed in Table 6-26 and outlined in the section below. BMP sizing worksheets are presented in Appendix E.

Table 6-26: Sand Filter Design Criteria

Design Parameter	Unit	Design Criteria
Stormwater quality design volume, SQDV	acre-feet	See Section 2 and Appendix E for calculating SQDV.
Max depth at SQDV	feet	3
Freeboard (minimum)	feet	1
Length to width ratio	L:W	2:1 (larger preferred)
Filter bed depth	inches	18 inches sand; 9 inches gravel
Max ponding depth above filter bed	feet	6
Drawdown time	Hours	?

Design Parameter	Unit	Design Criteria
Hydraulic conductivity of sand, k	in/hr	1 (equal to 2 ft/day)
Underdrains		6 inch minimum diameter; 0.5% minimum slope
Side slopes	H:V	4:1 (H:V) interior and 3:1 (H:V) exterior, unless stabilization has been approved by a licensed geotechnical engineer; or vertical concrete walls

Pretreatment

Pretreatment must be provided for sand filters in order to reduce the sediment load entering the filter. Pretreatment refers to design features that provide settling of large particles before runoff reaches the filter, easing the long-term maintenance burden. To ensure that pretreatment mechanisms are effective, designers shall incorporate pretreatment such as a biofiltration BMP, proprietary device, or sedimentation forebay. BMPs that are described in the 2011 TGM that may serve this purpose include:

For design specification of selected pre-treatment devices, refer to:

- VEG-3: Vegetated swale
- VEG-4: Vegetated filter strip
- PROP-1: Hydrodynamic separation device

Sizing Criteria

Background

Sand filter design is based on Darcy's law:

$$Q = KiA \qquad \text{(Equation 6-104)}$$

Where:

Q	=	water quality design flow (cfs)
K	=	hydraulic conductivity (fps)
A	=	surface area perpendicular to the direction of flow (ft^2)
i	=	hydraulic gradient (ft/ft) for a constant head and constant media depth, computed as follows:

$$i = \frac{h+l}{l} \qquad \text{(Equation 6-105)}$$

Where:

h	=	average depth of water above the filter (ft), defined for this design as $d/2$
d	=	maximum storage depth above the filter (ft)
l	=	thickness of sand media (ft)

Darcy's law underlies both the simple and the routing methods of design. The filtration rate V, or more correctly, $1/V$, is the direct input in the sand filter design. The relationship between the filtration rate V and hydraulic conductivity K is revealed by equating Darcy's law and the equation of continuity, $Q = VA$. Specifically:

$Q = KiA$ and $Q = VA$

So, $VA = KiA$

Or: $V = Ki$ (Equation 6-106)

Where,

V	=	filtration rate (ft/s)

Note that $V \neq K$. That is, the filtration rate is not the same as the hydraulic conductivity, but they do have the same units (distance per time). K can be equated to V by dividing V by the hydraulic gradient i, which is defined above.

The hydraulic conductivity K does not change with head nor is it dependent on the thickness of the media, only on the characteristics of the media and the fluid. A design hydraulic conductivity of 1 inch per hour (2 feet per day) used in this simple sizing method is based on bench-scale tests of conditioned rather than clean sand (KCSWDM, 2005) and represents the average sand bed condition as silt is captured and held in the sand bed.

Unlike the hydraulic conductivity, the filtration rate V changes with head and media thickness, although the media thickness is constant in the sand filter design.

Simple Sizing Method

The simple sizing method does not route flows through the filter. It determines the size of the filter based on the simple assumption that inflow is immediately discharged through the filter as if there were no storage volume. An adjustment factor (0.7) is applied to compensate for the greater filter size resulting from this method. Even with the adjustment factor, the simple method generally produces a larger filter size than the routing method.

Step 1: Determine the water quality design volume

Sand filters should be sized to capture and treat the stormwater quality design volume (see Section E.1).

Step 2: Determine maximum storage depth of water

Determine the maximum water storage depth (d) above the sand filter. This depth is defined as the depth at which water begins to overflow the reservoir pond, and it depends on the site topography and hydraulic constraints. The depth is chosen by the designer, but should be 6 feet or less.

Step 3: Calculate the sand filter area

Determine the sand filter area using the following equation:

$$A_{sf} = \frac{V_{wq} R L}{K t (h + L)} \qquad \text{(Equation 6-107)}$$

Where,

A_{sf}	=	surface area of the sand filter bed (ft^2)
V_{wq}	=	water quality design volume (ft^3)
R	=	routing adjustment factor (use $R = 0.7$)
L	=	sand bed depth (ft)
K_{des}	=	design hydraulic conductivity of media (use 2 ft/day)
t	=	drawdown time (use 1 day)
h	=	average depth of water above the filter (ft), [use (d/2) with d from Step 2]

Routing Method

A continuous runoff model, such as US EPA's Stormwater Management Model (SWMM) Model, can be used to optimally size a sand filter. A continuous simulation model consists of three components: a representative long term period of rainfall data ($\approx$ 20 years or greater) as the primary model input; a model component representing the tributary area to the sand filter that takes into account the amount of impervious area, soil types of the pervious area, vegetation, evapotranspiration, etc.; and a component that simulates the sand filter. Using this method, the filter should be sized to capture and treat the WQ design volume from the post-development tributary area.

The continuous simulation model routes predicted tributary runoff to the sand filter, where treatment is simulated as a function of the infiltrative (flow) capacity of the sand filter and the available storage volume above the sand filter. In a continuous runoff model such as SWMM, the physical parameters of the sand filter are represented with stage-storage-discharge relationships. Due to the computational power of ordinary desktop computers, long-term continuous simulations generally take only minutes to run. This allows the modeler to run several simulations for a range of sand filter sizes, varying either the surface area of the filter (and resulting flow capacity) or the storage capacity above the sand filter, or both. Sufficient continuous model simulations should be completed so that results encompass the WQ design volume capture goal.

Model results should be plotted for both varying storage depths above the filter and for varying filter surface area (and resulting flow capacity) while keeping all other parameters constant. The resulting relationship of percent capture as a function of sand filter flow and storage capacity can be used to optimally size a sand filter based on site conditions and restraints.

In addition to continuous simulation modeling, routing spreadsheets and/or other forms of routing modeling that incorporate rainfall-runoff relationships and infiltrative (flow) capacities of sand filters may be used to size facilities. Alternative sizing methodologies should be prepared with good engineering practices.

Sizing and Geometry

1) Sand filters shall be sized to capture and filter the Stormwater quality design volume, SQDV (See Section 2 and Appendix E for further detail).

2) Sand filters may be designed in any geometric configuration, but rectangular with a 2:1 length-to-width ratio or greater is preferred.

3) Filter bed depth must be at least 24 inches, but 36 inches is preferred.

4) Depth of water storage over the filter bed should be 6 feet maximum. Minimum freeboard is one foot.

5) Sand filters should be placed off-line to prevent scouring of the filter bed by high flows. The overflow structure must be designed to pass the stormwater quality design storm.

Sand Specification

Ideally the effective diameter of the sand, d_{10} (the diameter corresponding to the sieve size that passes 10% of sand grains), should be just small enough to ensure a good quality effluent while preventing penetration of stormwater particles to such a depth that they cannot be removed by surface scraping (~2-3 inches). This effective diameter usually lies in the range 0.20-0.35 mm. In addition, the coefficient of uniformity, Cu = d_{60}/d_{10}, should be less than 3.

The sand in a filter should consist of medium sand with few fines meeting ASTM C 33 size gradation (by weight) or equivalent as given in the table below.

U.S. Sieve Size	Percent Passing
3/8 inch	100
U.S. No. 4	95 to 100
U.S. No. 8	80 to 100
U.S. No. 16	50 to 85
U.S. No. 30	25 to 60
U.S. No. 50	5 to 30
U.S. No. 100	Less than 10

Finally, the silica (SiO_2) content of the sand should be greater than 95% by weight.

Underdrain

1) There are several underdrain system options which can be used in the design of a sand filter:

    a. A central underdrain collection pipe with lateral collection pipes in an 8 inch minimum gravel backfill or drain rock bed.

    b. Longitudinal pipes in an 8 inch minimum gravel backfill or drain rock bed, with a collection pipe at the outfall.

    c. Small sand filters may use a single underdrain pipe in an 8 inch minimum gravel backfill or drain rock bed.

2) All underdrain pipes and connectors should be 6 inches or greater so they can be cleaned without damage to the pipe. Clean-out risers with diameters equal to the underdrain pipe should be placed at the terminal ends of all pipes and extend to the surface of the filter. A valve box should be provided for access to the cleanouts and the cleanout assembly should be water tight to prevent short circuiting of the sand filter.

3) The underdrain pipe should be sized and perforated as to ensure free draining of the sand filter bed. Round perforations should be at least 1/2-inch in diameter and the pipe should be laid with holes downward.

4) The maximum perpendicular distance between any two lateral collection pipes or from the edge of the filter and the collection pipes should be 9 feet.

5) All pipes should be placed with a minimum slope of 0.5%.

6) The invert of the underdrain outlet should be above the seasonal high groundwater level.

7) At least 8 inches of gravel backfill should be maintained over all underdrain piping, and at least 6 inches should be maintained on both side and beneath the pipe to prevent damage by heavy equipment during maintenance. Either drain rock or gravel backfill may be used between pipes.

8) The bottom gravel layer should have a diameter at least 2X the size of the openings into the drainage system. The grains should be hard, preferably rounded, with a specific gravity of at least 2.5, and free of clay, debris and organic impurities.

9) Either a geotextile fabric or a two-inch transition gradation layer (preferred) should be placed between the sand layer and the drain rock or gravel backfill layer. If a geotextile is used, one inch of drain rock or gravel backfill should be place above the fabric. This allows for a transitional zone between sand and gravel and may reduce pooling of water at the liner interface. The geotextile should meet the following minimum materials requirements.

Geotextile Property	Value	Test Method
Trapezoidal Tear (lbs)	40 (min)	ASTM D4533
Permeability (cm/sec)	0.2 (min)	ASTM D4491
AOS (sieve size)	#60 - #70 (min)	ASTM D4751
Ultraviolet resistance	70% or greater	ASTM D4355

Flow Spreader

1) A flow spreader should be installed at the inlet along one side of the filter to evenly distribute incoming runoff across the filter and to prevent erosion of the filter surface.

    a. If the sand filter is curved or an irregular shape, a flow spreader should be provided for a minimum of 20 percent of the filter perimeter.

    b. If the length-to-width ratio of the filter is 2:1 or greater, a flow spreader should be located on the longer side and for a minimum length of 20 percent of the facility perimeter.

    c. In other situations, use good engineering judgment in positioning the spreader.

2) Erosion protection should be provided along the first foot of the sand bed adjacent to the flow spreader. Geotextile weighted with sand bags at 15-foot intervals may be used. Quarry spalls may also be used.

Vegetation

1) The use of vegetation in sand filters is optional. However, no top soil should be added to the sand filter bed because the fine-grained materials (silt and clay) would reduce the hydraulic capacity of the filter.

2) Growing grass or other vegetation requires the selection of species that can tolerate the demanding environment of a sand filter bed. Plants not receiving sufficient dry weather flows should be able to withstand long periods of drought during summer periods, followed by periods of saturation during storm events. A horticultural specialist should be consulted for advice on species selection.

3) A sod grown in sand may be used on the sand surface as long as there is no clay in the sand substrate and the particle size gradation of the substrate meets the sand filter specifications. No other sod should be used due to the high clay content in most sod soils.

4) To prevent uses that could compact and damage the filter surface, permanent structures are not permitted on sand filters (e.g. playground equipment).

Emergency Overflow Structure

Sand filters may only be placed off-line, but an emergency overflow must still be provided in the event the filter becomes clogged. The overflow structure must be able to safely convey flows from the stormwater quality design storm to the downstream conveyance system or other acceptable discharge point.

Side Slopes

1) Interior side slopes above the stormwater quality design depth and up to the emergency overflow water surface steeper than 4:1 (H:V) should be stabilized to prevent erosion with a method approved by the local permitting authority.

2) Exterior side slopes steeper than 2:1 (H:V) should be stabilized to prevent erosion with a method approved by the local permitting authority.

3) For any slope (interior or exterior) greater than 2:1 (H:V), a geotechnical investigation and report must be submitted and approved by the local permitting authority.

4) Pond walls may be vertical retaining walls, provided: (a) they are constructed of reinforced concrete, (b) a fence, which prevents access, is provided along the top of the wall or further back, and (c) the design is stamped by a licensed civil engineer and approved by the County.

Embankments

1) Embankments (earthen slopes or berms) may be used for detaining or redirecting the flow of water.

2) The minimum top width of all berm embankments should be 20 feet, or as approved by the geotechnical engineer.

3) Basin berm embankments should be constructed on native consolidated soil (or adequately compacted and stable fill soils analyzed by a licensed geotechnical engineer) free of loose surface soil materials, roots, and other organic debris.

4) Earthworks should be in accordance with Section 300-6 of the Standard Specifications for Public Works Construction, most recent edition.

5) Basin berm embankments greater than 4 feet in height should be constructed by excavating a key equal to 50% of the berm embankment cross-sectional height and width. This requirement may be waived if specifically recommended by a licensed geotechnical engineer.

6) The berm embankment should be constructed of compacted soil (95% minimum dry density, modified proctor method per ASTM D1557), placed in 6-inch lifts.

Maintenance Access

Maintenance access road(s) shall be provided to the control structure and other drainage structures associated with the basin (e.g., inlet, emergency overflow or bypass structures). Manhole and catch basin lids should be in or at the edge of the access road.

An access ramp is required for removal of sediment with a backhoe or loader and truck. The ramp should extend to the bottom of the sand filter.

Landscaping Outside of the Facility

A sand filter can add aesthetics to a site and should be incorporated into a project's landscape design. Interior side slopes may be stepped with flat areas to provide informal seating with a game or play area below. Perennial beds may be planted above the overflow water surface elevation. Large shrubs and trees are not recommended, however, as shading limits evaporation and falling leaves can clog the filter surface. If a sand filter area is intended for recreational uses, such as a volleyball area, the interior side slopes of the filter embankment should be no steeper than 3:1 and may be stepped.

1) No trees or shrubs may be planted within 10 feet of inlet or outlet pipes or manmade drainage structures such as spillways, flow spreaders, or earthen embankments. Species with roots that seek water, such as willow or poplar, should not be used within 50 feet of pipes or manmade structures.

2) Prohibited non-native plant species will not be permitted. For more information on invasive weeds, including biology and control of listed weeds, look at the

encycloweedia located at the California Department of Food and Agriculture website at or the California Invasive Plant Council website at www.cal-ipc.org.

Operations and Maintenance

Sand filters are subject to clogging by fine sediment, oil and grease, and other debris (e.g., trash and organic matter such as leaves). Filters and pretreatment facilities should be inspected every 6 months during the first year of operation. Inspection should also occur immediately following a storm event to assess the filtration capacity of the filter. Once the filter is performing as designed, the frequency of inspection may be reduced to once per year.

Most of the maintenance should be concentrated on the pretreatment practices, such as buffer strips and swales upstream of the trench to ensure that sediment does not reach the infiltration trench. Regular inspection should determine if the sediment removal structures require preventative maintenance.

Inspect basin a minimum of twice a year, before and after the rainy season, after large storm events, or more frequently if needed. Some important items to check for include: differential settlement, cracking; erosion, leakage, or tree growth on the embankment; the condition of the riprap in the inlet, outlet and pilot channels; sediment accumulation in the basin; and the vigor and density of the vegetation on the basin side slopes and floor. Correct observed problems as necessary.

- Remove litter and debris from banks and basin bottom as required.
- Repair erosion to banks and bottom as required.
- Check infiltration rate of sand bed twice annually, once after significant rainfall.
- Scarify top 3 to 5 inches of filters surface by raking once annually or as required to restore infiltration rate of the filter.
- Clean forebay every two years at a minimum, to avoid accumulation in main basin.
- Inspect outlet for clogging a minimum of twice a year, before and after the rainy season, after large storms, and more frequently if needed. Correct observed problems as necessary.

TCM-5: Cartridge Media Filter

Cartridge media filters are manufactured devices that typically consist of a series of cylindrical vertical filters contained in a catch basin, manhole, or vault that provide treatment through filtration and sedimentation. The manhole or vault may be divided into multiple chambers where the first chamber acts as a pre-settling basin for removal of coarse sediment while another chamber acts as the filter bay and houses the filter cartridges.

Cartridge Media Filters

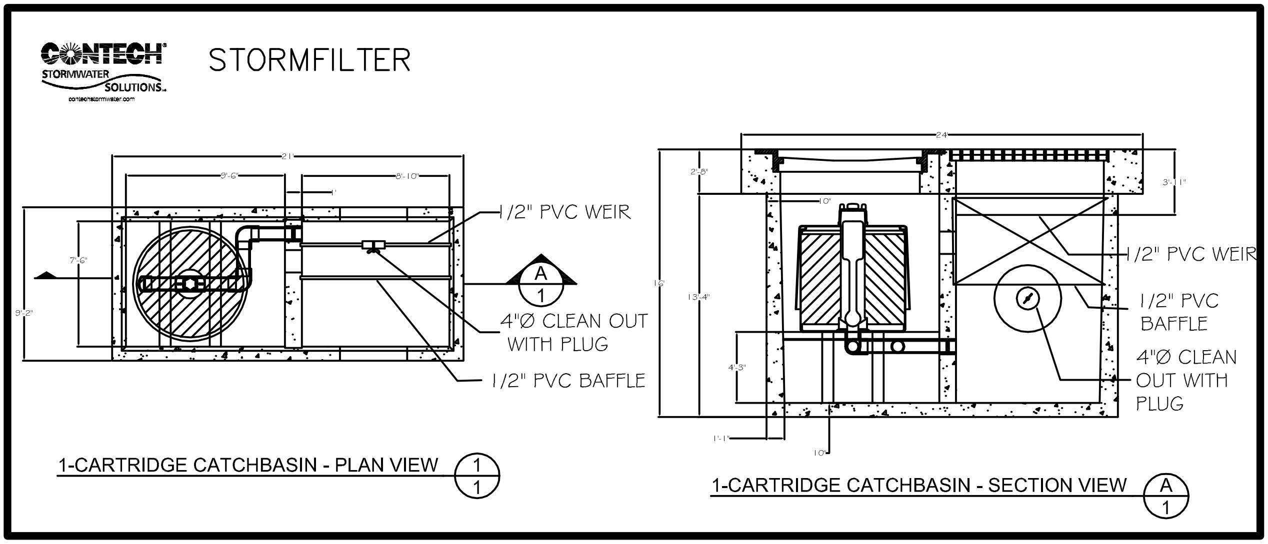
Photo Credits: Contech Stormwater Solutions, Inc.

Application

- Parking lots
- Roadways
- Playgrounds
- Outdoor eating areas

Preventative Maintenance

- Filter media replacement
- Solids removal from vault, manhole, or catch basin
- Inspect for inlet and outlet for clogging



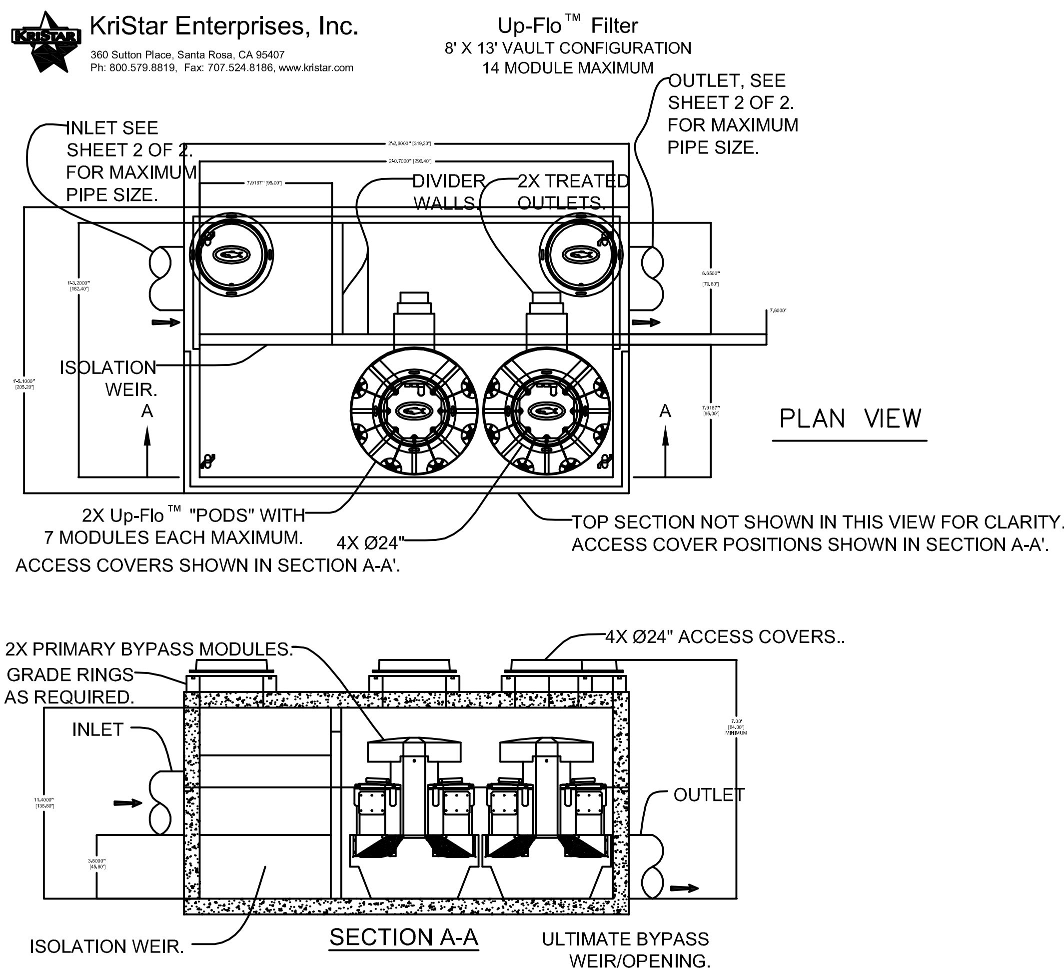


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Figure 6-24: Cartridge Media Filter

Table 6-27: Proprietary Cartridge Media Filter Manufacturer Websites

Device	Manufacturer	Website
BaySaver BayFilter	Baysaver Technologies Inc.	www.baysaver.com
ConTech StormFilter™	Contech® Construction Products Inc.	www.contech-cpi.com
CrystalStream	CrystalStream Technologies	www.crystalstream.com
KriStar Fossil Tee™ (media filter)	KriStar Enterprises Inc.	www.kristar.com
KriStar Up-Flo™ Filter and Perk™ Filter	KriStar Enterprises Inc.	www.kristar.com

Limitations

As with all filtration systems, use in catchments that have significant areas of non-stabilized soils can lead to premature clogging.

Design Criteria

1) Cartridge media filter BMP vendors are constantly updating and expanding their product lines, so refer to the latest design guidance from each of the vendors.

2) Selected filter media should target pollutants of concern. A combination of media is often recommended to maximize pollutant removal. Perlite is effective for removing TSS and oil and grease. Zeolite removes soluble metals, ammonium, and some organics. Vendors also offer proprietary medias (such as leaf compost or activated carbon) that are designed to remove soluble metals, organics, and other pollutants.

3) Manufacturers try to distinguish their products through innovative designs that aim at providing self cleaning and draining, uniformly loaded, and clog resistant cartridges that functional properly over a wide range of hydraulic loadings and pollutant concentrations.

4) All stormwater vaults containing cartridge filters that have standing water for longer than 72 hours can become a breeding area for mosquitoes. The selected BMP should have a system to completely drain the vault, such as weep holes in the bottom of the vault.

Sizing

1) Cartridge media filters should be sized to capture and treat the stormwater quality design flow rate.

2) Proprietary cartridge media filter devices, like most proprietary BMPs, and auxiliary components such as media, screens, baffles, and sumps are selected based onsite-specific conditions such as the loading that is expected and the desired frequency of maintenance. Sizing of proprietary devices is reduced to a simple process whereby a model can simply be selected from a table or a chart based on a few known quantities

(tributary area, location, design flow rate, etc). Most of the manufacturers either size the devices for potential clients or offer calculators on their websites that simplify the design process. For the latest sizing guidelines, refer to the manufacturer's website.

PT-1: Hydrodynamic Separation Device

Hydrodynamic separation devices (alternatively, swirl concentrators) are devices that remove trash, debris, and coarse sediment from incoming flows using screening, gravity settling, and centrifugal forces generated by forcing the influent into a circular motion. By having the water move in a circular fashion, rather than a straight line, it is possible to obtain significant removal of suspended sediments and attached pollutants with less space as compared to wet vaults and other settling devices. Hydrodynamic devices were originally developed for combined sewer overflows (CSOs), where they were used primarily to remove coarse inorganic solids. Hydrodynamic separation has been adapted for stormwater treatment by several manufacturers and is currently used to remove trash, debris, and other coarse solids down to sand-sized particles. Several types of hydrodynamic separation devices are also designed to remove floating oils and grease using sorbent media.

Hydrodynamic Separation

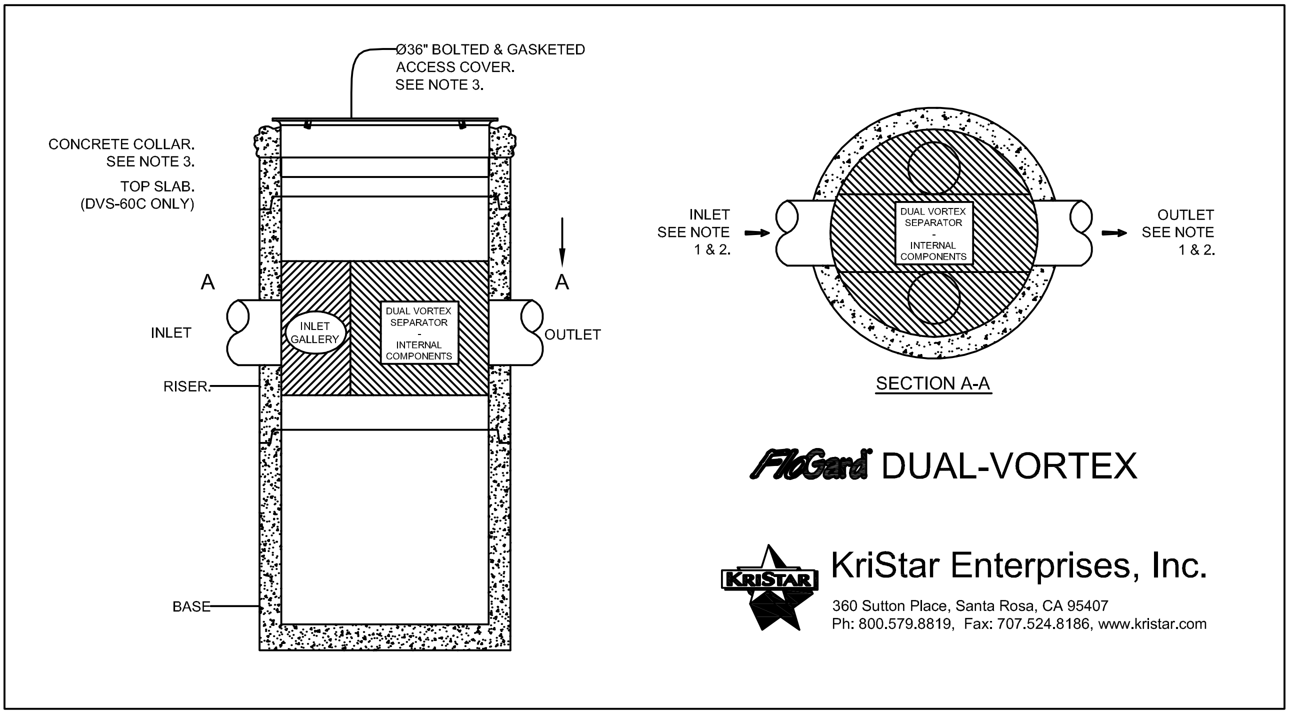
Photo Credits: 1. Contech Stormwater Solutions, Inc.;
2. Dave Weller, FedCo Construction

Application

- Parking lots
- Areas adjacent to parking lots
- Areas adjacent to buildings
- Road medians and shoulders

Preventative Maintenance

- Sediment, trash and debris removal
- Vector control



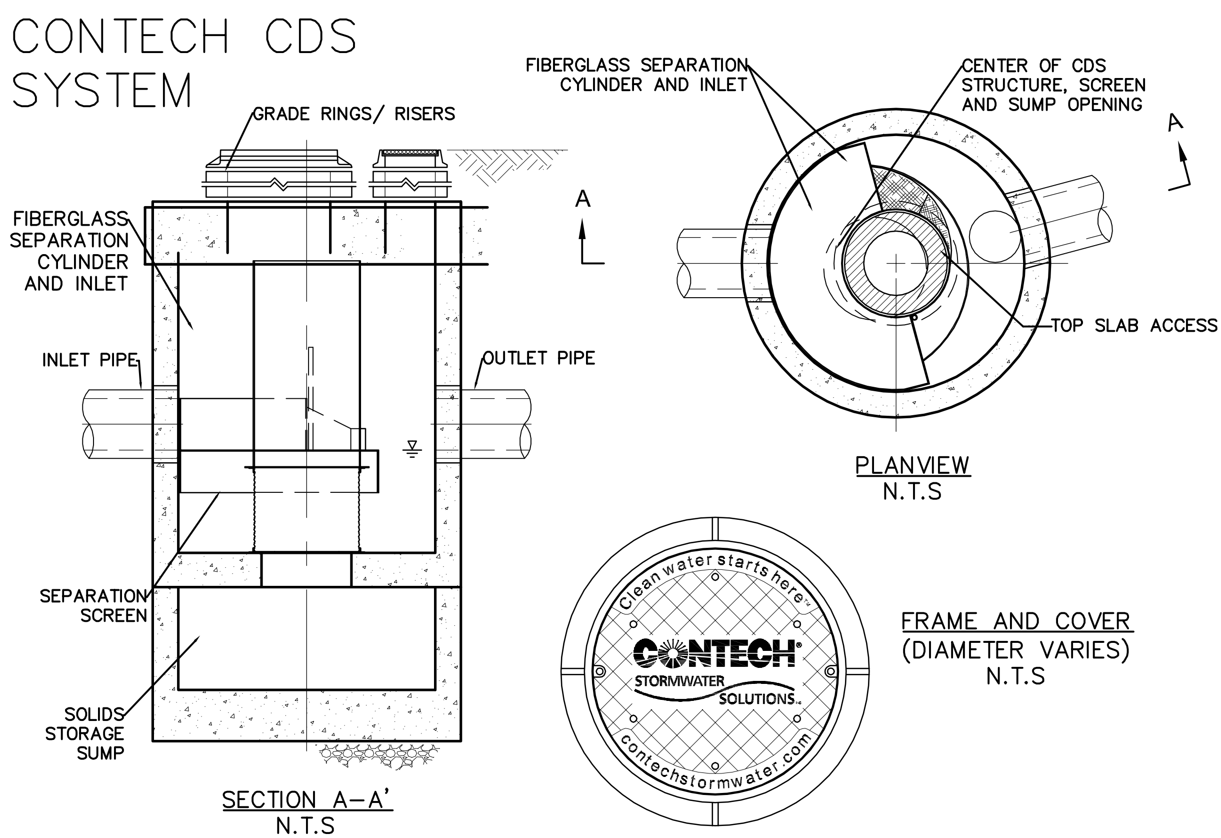


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Figure 6-25: Hydrodynamic Separation

Table 6-28: Proprietary Hydrodynamic Device Manufacturer Websites

Device	Manufacturer	Website
Rinker In-Line Stormceptor®	Rinker Materials™	www.rinkerstormceptor.com
FloGard® Dual-Vortex Hydrodynamic Separator	KriStar Enterprises Inc.	www.kristar.com
Contech® CDSª™	Contech® Construction Products Inc.	www.contech-cpi.com
Contech® Vortechs™	Contech® Construction Products Inc.	www.contech-cpi.com
Contech® VorSentry™	Contech® Construction Products Inc.	www.contech-cpi.com
Contech® VorSentry™ HS	Contech® Construction Products Inc.	www.contech-cpi.com
BaySaver BaySeparator	Baysaver Technologies Inc.	www.baysaver.com

Limitations

Hydrodynamic separation devices are effective for the removal of course sediment, trash, and debris, and are useful as pretreatment in combination with other BMP types that target smaller particle sizes.

Hydrodynamic devices represent a wide range of device types that have different unit processes and design elements (e.g., storage versus flow-through designs, inclusion of media filtration, etc.) that vary significantly within the category. These design features likely have significant effects on BMP performance; therefore, generalized performance data for hydrodynamic devices is not practical.

Design Criteria

Proprietary hydrodynamic device BMP vendors are constantly updating and expanding their product lines, so refer to the latest design guidance from each of the vendors. General guidelines on the performance, sizing, operations and maintenance of proprietary devices are provided by the vendors.

Sizing

Hydrodynamic devices shall be sized to capture and treat the stormwater quality design flow rate and to completely drain within 72 hours.

Sizing of proprietary devices is reduced to a simple process whereby a model can simply be selected from a table or a chart based on a few known quantities (tributary area, location, design flow rate, design volume, etc). A few of the manufacturers either size the devices for potential clients or offer calculators on their websites that simplify the design process even further and lessens the possibility of using obsolete design information. For the latest sizing guidelines, refer to the manufacturer's website.

The hydrodynamic separators listed in Table 6-28 are designed to have a permanent pool of water stored within the system. Various methods of vector control are available to prevent mosquito breeding including manhole cover screens and the use of mosquito dunks. In many designs, oil and grease is stored at the water surface and provides a deterrent to mosquito breeding.

Operations and Maintenance

Hydrodynamic devices should be inspected every 6 months during the first year of operation. Inspection should also occur immediately following a storm event to assess the function of the device. Once the device is performing as designed, the frequency of inspection may be reduced to once per year.

PT-2: Catch Basin Insert

Catch basin inserts are manufactured filters or fabric placed in a drop inlet to remove sediment and debris and may include sorbent media (oil absorbent pouches) to remove floating oils and grease. Catch basin inserts are selected specifically based upon the orientation of the inlet.

Catch Basin Inserts

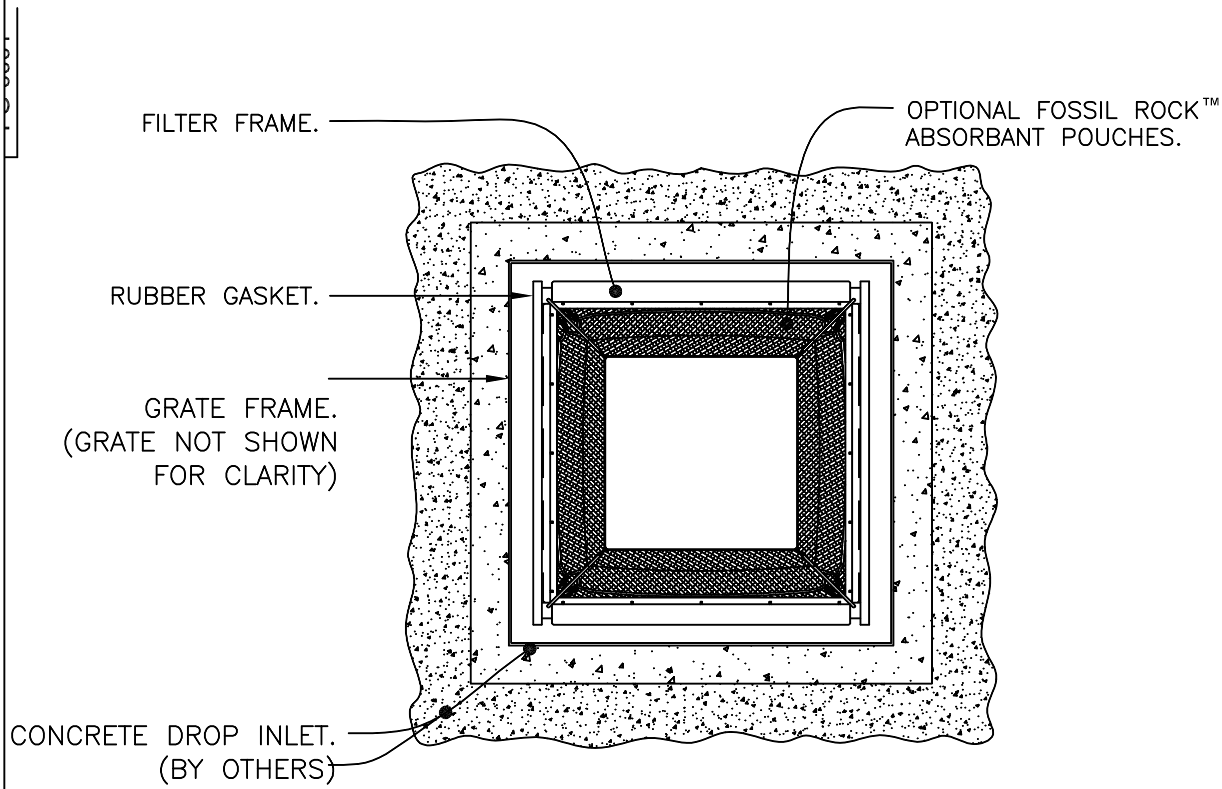
Photo Credits: 1. KriStar; 2. Aquashield

Application

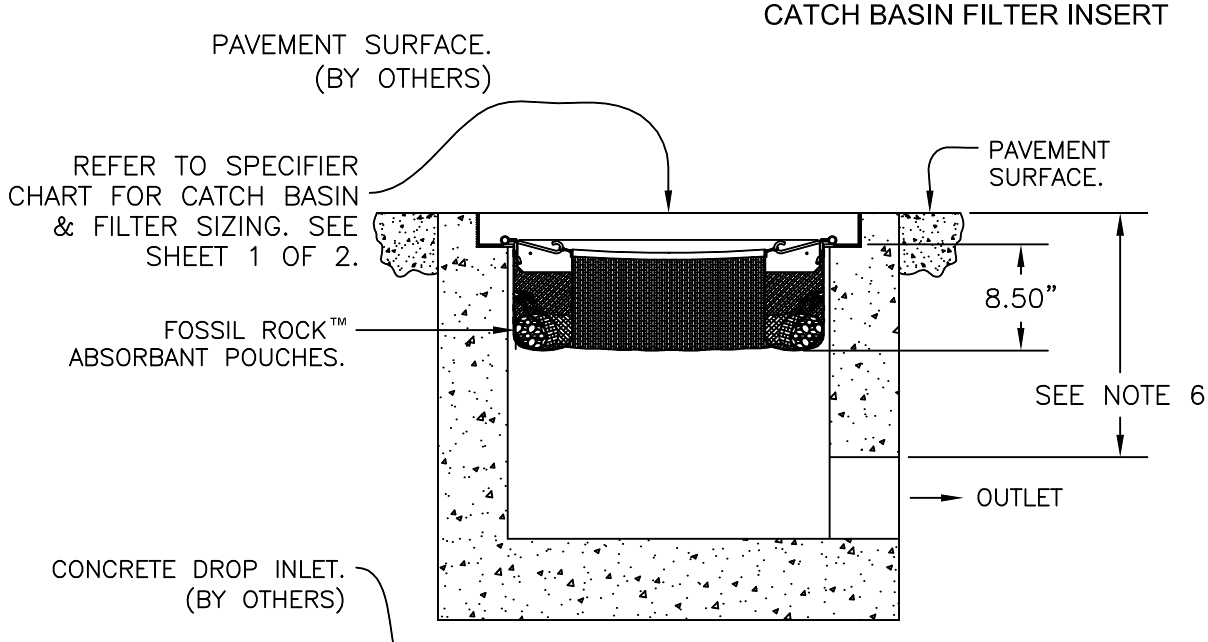
- Parking lots
- Roads
- Athletic courts
- Outdoor food areas

Preventative Maintenance

- After storm inspection
- Sediment removal
- Trash removal
- Filter/sorbent media replacement



CATCH BASIN FILTER INSERT



SECTION VIEW

KriStar Enterprises, Inc.
360 Sutton Place, Santa Rosa, CA 95407
Ph: 800.579.8819, Fax: 707.524.8186, www.kristar.com

FloGard®

Geosyntec consultants

Figure 6-26: Catch Basin

Table 6-29: Proprietary Catch Basin Insert Manufacturer Websites

Device	Manufacturer	Website
AbTech Industries Ultra-Urban Filter™	AbTech Industries	www.abtechindustries.com
Aquashield Aqua-Guardian™ Catch Basin Insert	Aquashield™ Inc.	www.aquashieldinc.com
Bowhead StreamGuard™	Aquashield™ Inc.	www.aquashieldinc.com
Contech® Triton Catch Basin Filter™	Contech® Construction Products Inc.	www.contech-cpi.com
Contech® Triton Curb Inlet Filter™	Contech® Construction Products Inc.	www.contech-cpi.com
Contech® Triton Basin StormFilter™	Contech® Construction Products Inc.	www.contech-cpi.com
Contech® Curb Inlet StormFilter™	Contech® Construction Products Inc.	www.contech-cpi.com
Curb Inlet Basket	SunTree Technologies Inc.	www.suntreetech.com
Curb Inlet Grates	EcoSense International™	www.ecosenseinternational.org
Grate Inlet Skimmer Box	SunTree Technologies Inc.	www.suntreetech.com
Hydro-Kleen™ Filtration System	Hydro Compliance Management Inc.	Not available
KriStar FloGard+PLUS®	KriStar Enterprises Inc.	www.kristar.com
KriStar FloGard®	KriStar Enterprises Inc.	www.kristar.com
KriStar FloGard LoPro Matrix Filter®	KriStar Enterprises Inc.	www.kristar.com
Nyloplast Storm-PURE Catch Basin Insert	Nyloplast Engineered Surface Drainage Products	www.nyloplast-us.com
StormBasin®	FabCo® Industries Inc.	www.fabco-industries.com
Stormdrain Solutions Interceptor	FabCo® Industries Inc.	www.fabco-industries.com
Stormdrain Solutions Inceptor®	Stormdrain Solutions	www.stormdrains.com
StormPod®	FabCo® Industries Inc.	www.fabco-industries.com
Stormwater Filtration Systems	EcoSense International™	www.ecosenseinternational.org
Ultra-CurbGuard®	UltraTech International Inc.	www.spillcontainment.com
Ultra-DrainGuard®	UltraTech International Inc.	www.spillcontainment.com
Ultra-GrateGuard®	UltraTech International Inc.	www.spillcontainment.com
Ultra-GutterGuard®	UltraTech International Inc.	www.spillcontainment.com
Ultra-InletGuard®	UltraTech International Inc.	www.spillcontainment.com

Limitations

Catch basin inserts come in such a wide range of configurations that it is practically impossible to generalize the expected performance. Inserts should mainly be used for catching coarse sediments and floatable trash, and are effective as pretreatment in combination with other types of structures that are recognized as water quality treatment BMPs. Trash and large objects can greatly reduce the effectiveness of catch basin inserts with respect to sediment and hydrocarbon capture. Frequent maintenance and the use of screens and grates to keep trash out may decrease the likelihood of clogging and prevent obstruction and bypass of incoming flows.

Design Criteria

Catch basin inserts shall be sized to capture and treat the stormwater quality design flow rate.

Operations and Maintenance

1) Trash, debris, and sediment around insert grate and inside chamber requiring trash to be cleared.

2) Repair filter media if damaged or severely clogged.

3) Inspection of catch basin insert after each storm greater than 0.2 inches is recommended.

7 MAINTENANCE PLAN

This chapter identifies the basic information that should be included in a maintenance plan. Refer to Fact Sheets for individual control measures in Chapter 6 regarding device-specific maintenance requirements.

7.1 Site Map

1) Provide a site map showing boundaries of the site, acreage and drainage patterns/contour lines. Show each discharge location from the site and any drainage flowing onto the site. Distinguish between soft and hard surfaces on the map.

2) Identify locations of existing and proposed storm drain facilities, private sanitary sewer systems and grade-breaks for purposes of pollution prevention.

3) With legend, show locations of expected sources of pollution generation (outdoor work and storage areas, heavy traffic areas, delivery areas, trash enclosures, fueling areas, industrial clarifiers, wash-racks, etc). Identify any areas having contaminated soil or where toxins are stored or have been stored/disposed of in the past.

4) With legend, indicate types and locations of stormwater management control measures which will be built to permanently control stormwater pollution. Distinguish between pollution prevention, treatment, sewer diversion, and containment devices.

7.2 Baseline Descriptions

1) List the property owners and persons responsible for operation and maintenance of the stormwater management control measures onsite. Include phone numbers and addresses.

2) Identify the intended method of providing financing for operation, inspection, routine maintenance and upkeep of stormwater control measures.

3) List all permanent stormwater control measures. Provide a brief description of stormwater management control measures selected and if appropriate, facts sheets or additional information.

4) As appropriate for each stormwater control measure provide:

    a. A written description and check list of all maintenance and waste disposal activities that will be performed. Distinguish between the maintenance appropriate for a 2-year establishment period and expected long-term maintenance. For example, maintenance requirements for vegetation in a constructed wetland may be more intensive during the first few years until the vegetation is established. The post-establishment maintenance plan should address maintenance needs (e.g., pruning, irrigation, weeding) for a larger, more stable system. Include maintenance performance procedures for facility components that require relatively unique maintenance knowledge, such as specific plant removal / replacement, landscape features, or constructed wetland maintenance. These procedures should provide enough detail for a person unfamiliar with maintenance to perform the activity, or identify the specific skills or knowledge necessary to perform and document the maintenance.

b. A description of site inspection procedures and documentation system, including record-keeping and retention requirements.

c. An inspection and maintenance schedule, preferably in the form of a table or matrix, for each activity for all facility components. The schedule should demonstrate how it will satisfy the specified level of performance, and how the maintenance / inspection activities relate to storm events and seasonal issues.

d. Identification of the equipment and materials required to perform the maintenance.

5) As appropriate, list all housekeeping procedures for prohibiting illicit discharges or potential illicit discharges to the storm drain. Identify housekeeping BMPs that reduce maintenance of Treatment Control Measures. These procedures are listed based on facility operations and can be found in the Ventura County Industrial/Commercial Clean Business Program document.

7.3 Spill Plan

1) Provide emergency notification procedures (phone and agency/persons to contact)

2) As appropriate for site, provide emergency containment and cleaning procedures.

3) Note downstream receiving water bodies or wetlands which may be affected by spills or chronic untreated discharges.

4) As appropriate, create an emergency sampling procedure for spills. (Emergency sampling can protect the property owner from erroneous liability for down-stream receiving area clean-ups).

7.4 Facility Changes

Operational or facility changes which significantly affect the character or quantity of pollutants discharging into the stormwater management control measures will require modifications to the Maintenance Plan and/or additional stormwater control measures.

7.5 Training

1) Identify appropriate persons to be trained and assure proper training.

2) Training to include:

   a. Good housekeeping procedures defined in the plan.

   b. Proper maintenance of all pollution mitigation devices.

   c. Identification and cleanup procedures for spills and overflows.

   d. Large-scale spill or hazardous material response.

   e. Safety concerns when maintaining devices and cleaning spills.

7.6 Basic Inspection and Maintenance Activities

1) Create and maintain onsite, a log for inspector names, dates and stormwater control measure devices to be inspected and maintained. Provide a checklist for each inspection and maintenance category.

2) Once annually, perform testing of any mechanical or electrical devices prior to wet weather.

3) Report any significant changes in stormwater management control measures to the site management. As appropriate, assure mechanical devices are working properly and/or landscaped BMP plantings are irrigated and nurtured to promote thick growth.

4) Note any significant maintenance requirements due to spills or unexpected discharges.

5) As appropriate, perform maintenance and replacement as scheduled and as needed in a timely manner to assure stormwater management control measures are performing as designed and approved.

6) Assure unauthorized low-flow discharges from the property do not by-pass stormwater control measures.

7) Perform an annual assessment of each pollution generation operation and its associated stormwater management control measures to determine if any part of the pollution reduction train can be improved.

7.7 Revisions of Pollution Mitigation Measures

If future correction or modification of past stormwater management control measures or procedures is required, the owner shall obtain approval from the governing stormwater agency prior to commencing any work. Corrective measures or modifications shall not cause discharges to bypass or otherwise impede existing stormwater control measures.

7.8 Monitoring & Reporting Program

1) The governing stormwater agency may require a Monitoring & Reporting Program to assure the stormwater management control measures approved for the site are performing according to design.

2) If required by local permitting agency, the Maintenance Plan shall include performance testing and reporting protocols.

APPENDIX A : ACRONYMS AND GLOSSARY OF TERMS

A.1 Acronyms and Abbreviations

303(d)	303(d) List of Impaired Water Bodies
API	American Petroleum Institute (oil/water separator type)
BMP	Best Management Practice
CEQA	California Environmental Quality Act
CP	Coalescing Plate (oil/water separator type)
CTR	California Toxics Rule
CWA	Clean Water Act
CDFG	California Department of Fish and Game
EIA	Effective Impervious Area
EMC	Event Mean Concentration
ESA	Environmentally Sensitive Area
LID	Low Impact Development
MEP	Maximum Extent Practicable
MS4	Municipal Separate Storm Sewer System
RPAMP	Redevelopment Project Area Master Plan
SQDV	Stormwater Quality Design Volume
SQDF	Stormwater Quality Design Flow
TSS	Total Suspended Solids
USACE	United States Army Corps of Engineers
USEPA	United States Environmental Protection Agency
WERF	Water Environment Research Foundation

A.2 Glossary

Automotive Repair Shop: A facility that is categorized in any one of the following Standard Industrial Classification (SIC) codes: 5013, 5014, 5541, 7532-7534, or 7536-7539.

Backfill: Earth or engineered material used to refill a trench or an excavation.

Berm: An earthen mound used to direct the flow of runoff around or through a structure.

Best Management Practice (BMP): Any program, technology, process, siting criteria, operational methods or measures, or engineered systems, which when implemented prevent, control, remove, or reduce pollution.

Best Management Practices (BMPs): Includes schedules of activities, prohibitions of practices, maintenance procedures, and other management practices to prevent or reduce the pollution of waters of the United States. BMPs also include treatment requirements, operating procedures, and practices to control plant site runoff, spillage or leaks, sludge or waste disposal, or drainage from raw material storage.

Biofiltration: The simultaneous process of filtration, infiltration, adsorption, and biological uptake of pollutants in stormwater that takes place when runoff flows over and through vegetated areas.

Bioretention Facility: A facility that utilizes soil infiltration and both woody and herbaceous plants to remove pollutants from stormwater runoff. Runoff is typically captured and infiltrated or released over a period of 24 to 48 hours.

Blue Roof: A roof that is designed to store rainwater, typically in a cistern-type device.

Brown Roof: A type of green roof which focuses on biodiversity and locally-sourced material.

Buffer Strip or Zone: Strip of erosion-resistant vegetation over which stormwater runoff is directed.

Capacity: The capacity of a stormwater drainage facility is the flow volume or rate that the facility (e.g., pipe, basin, vault, swale, ditch, drywell, etc.) is designed to safely contain, receive, convey, reduce pollutants from, or infiltrate stormwater to meet a specific performance standard. There are different performance standards for pollution reduction, flow control, conveyance, and destination/ disposal, depending on location.

Catch Basin: Box-like underground concrete structure with openings in curbs and gutters designed to collect runoff from streets and pavements.

Check Dam: Small temporary barrier, grade control structure, or dam constructed across a swale, drainage ditch, or area of concentrated flow with the intent to slow or stop runoff.

Clean Water Act (CWA): (33 U.S.C. 1251 et seq.) requirement of the National Pollutant Discharge Elimination System (NPDES) program are defined under Sections 307, 402, 318 and 405 of the CWA.

Commercial Development: Any development on private land that is not heavy industrial or residential. The category includes, but is not limited to: hospitals, laboratories and other medical facilities, educational institutions, recreational facilities, plant nurseries, multi-apartment buildings, car wash facilities, mini-malls and other business complexes, shopping malls, hotels, office buildings, public warehouses and other light industrial complexes.

Conduit: Any channel or pipe for directing the flow of water.

Construction General Permit: A NPDES permit issued by the State Water Resources Control Board (SWRCB) for the discharge of stormwater associated with construction activity from soil disturbance of five (5) acres or more.

Control Device: A device used to hold back or direct a calculated amount of stormwater to or from a stormwater management facility. Typical control structures include vaults or manholes fitted with baffles, weirs, or orifices.

Conveyance System: Any channel or pipe for collecting and directing the Stormwater.

Culvert: A covered channel or a large diameter pipe that crosses under a road, sidewalk, etc.

Dead-end Sump: A below surface collection chamber for small drainage areas that is not connected to the public storm drainage system. Accumulated water in the chamber must be pumped and disposed in accordance with all applicable laws.

Designated Public Access Points: Any pedestrian, bicycle, equestrian, or vehicular point of access to jurisdictional channels in the area of Ventura County subject to permit requirements.

Detention: The temporary storage of stormwater runoff to allow treatment by sedimentation and metered discharge of runoff at reduced peak flow rates.

Detention Facility: A facility designed to receive and hold stormwater and release it at a slower rate, usually over a number of hours. The full volume of stormwater that enters the facility is eventually released.

Detention Tank, Vault, or Oversized Pipe: A structural subsurface facility used to provide flow control for a particular drainage basin.

Development: any construction, rehabilitation, redevelopment or reconstruction of any public or private residential project (whether single-family, multi-unit or planned unit development); industrial, commercial, retail and any other non-residential projects, including public agency projects; or mass grading for future construction.

Directly Adjacent: Situated within 200 feet of the contiguous zone required for the continued maintenance, function, and structural stability of the environmentally sensitive area.

Directly Connected Impervious Area (DCIA): The area covered by a building, impermeable pavement, and/ or other impervious surfaces, which drains directly into the storm drain without first flowing across permeable land area (e.g. turf buffers).

Directly Discharging: Outflow from a drainage conveyance system that is composed entirely or predominantly of flows from the subject, property, development, subdivision, or industrial facility, and not commingled with the flows from adjacent lands.

Discharge: A release or flow of Stormwater or other substance from a conveyance system or storage container.

Disturbed Area: Any area that is altered as a result of land disturbance, such as: clearing, grading, grubbing, stockpiling and excavation.

Drainage Basin: A specific area that contributes stormwater runoff to a particular point of interest, such as a stormwater management facility, drainageway, wetland, river, or pipe.

Effective Impervious Area (EIA): That portion of the surface area that is hydrologically connected via sheet flow over a hardened conveyance or impervious surface without any intervening medium to mitigate flow volume.

Environmentally Sensitive Area (ESA): An area "in which plant or animal life or their habitats are either rare or especially valuable because of their special nature or role in an ecosystem and which would be easily disturbed or degraded by human activities and developments" (California Public Resources Code § 30107.5). Areas subject to stormwater mitigation requirements are: 303(d) listed water bodies in all reaches that are unimproved, all California Coastal Commission's *Environmentally Sensitive Habitat Areas* as delineated on maps in Local Coastal Plans, and Regional Water Quality Control Board's Basin Plan Rare, Threatened or Endangered Species (RARE) and Preservation of Biological Habitats (BIOL) designated waterbodies. The California Department of Fish and Game's (CDFG) *Significant Natural Areas* map will be considered for inclusion as the department field-verifies the designated locations. Watershed restoration projects will be considered for inclusion as the department field verifies the designated locations.

Erosion: The wearing a way of land surface by wind or water. Erosion occurs naturally from weather or runoff, but can be intensified by land-clearing practices relating to farming; residential, commercial, or industrial development; road building; or timber cutting.

Excavation: The process of removing earth, stone, or other materials, usually by digging.

Existing Urban Area: Existing urban areas and corresponding maps in Appendix B are based on the cities' City Urban Restriction Boundaries (CURB) lines and the Existing Community designation in the unincorporated County. These boundaries are a growth management tool intended to channel growth and protect agricultural and open-space land. The 2011 TGM utilizes existing urban areas (as defined in Appendix B) to provide parameters around eligibility for alternative compliance in two areas: 1) Smart Growth and 2) low income housing projects.

Extended Detention Basin: A surface vegetated basin used to provide flow control for a particular drainage basin. Stormwater temporarily fills the extended detention basin during large storm events and is slowly released over a number of hours, reducing peak flow rates.

Facility: Is a collection of industrial process discharging stormwater associated with industrial activity within the property boundary or operational unit.

Filter Fabric: Geotextile of relatively small mesh or pore size that is used to: (a) allow water to pass through while keeping sediment out (permeable); or (b) prevent both runoff and sediment from passing through (impermeable).

Filter Strip: A gently sloping, densely grassed area used to filter, slow, and infiltrate stormwater.

Flow Control Facility: Any structure or drainage device that is designed, constructed, and maintained to collect, retain, infiltrate, or detain surface water runoff during and after a storm event for the purpose of controlling post-development quantity leaving the site.

Flow Control: The practice of limiting the release of peak flow rates, flow durations, and volumes from a site. Flow control is intended to protect downstream properties, infrastructure, and natural resources from the increased stormwater runoff flow rates and volumes resulting from development.

Grading: The cutting and/or filling of the land surface to a desired shape or elevation.

Green Roof: A roofing system that layers a soil/vegetative cover over a waterproofing membrane. Green roofs rely on highly porous media and moisture retention layers to store intercepted precipitation and to support vegetation that can reduce the volume of stormwater runoff via evapotranspiration

Hazardous Substance: (1) Any material that poses a threat to human health and/or the environment. Typical hazardous substances are toxic, corrosive, ignitable, explosive, or chemically reactive; (2) Any substance named by EPA to be reported if a designated quantity of the substance is spilled in the waters of the United States or if otherwise emitted into the environment.

Hazardous Waste: By-products of society that can pose a substantial or potential hazard to human health or the environment when improperly managed. Possesses at least one of four characteristics (flammable, corrosivity, reactivity, or toxicity), or appears on special EPA lists.

Hillside: Property located in an area with known erosive soil conditions, where the development contemplates grading on any natural slope that is 25 percent or greater.

Hydrodynamic Separation: Flow-through structures with a settling or separation unit to remove sediments and other pollutants in which no outside power source is required, because the energy of the flowing water allows the sediments to efficiently separate. Depending on the type of unit, this separation may be by means of swirl action or indirect filtration.

Illegal Discharges: Any discharge to a municipal separate storm sewer that is not composed entirely of stormwater except discharges authorized by an NPDES permit (other than the NPDES permit for discharges from the municipal separate storm sewer) and discharges resulting from fire fighting activities.

Impervious Surface / Area: A hard surface area which either prevents or retards the entry of water into the predevelopment soil mantle. A hard surface area which causes water to run off the surface in greater quantities or at an increased rate of flow from the flow present under predevelopment conditions. Common impervious surfaces include, but are not limited to, roof tops, walkways, patios, driveways, parking lots or storage areas, (impermeable) concrete or asphalt paving, gravel roads, packed earthen materials, and oiled macadam or other surfaces which similarly impede the natural infiltration of storm water.

Industrial General Permit: A NPDES permit issued by the State Water Resources Control Board for the discharge of Stormwater associated with industrial activity.

Infiltration: The downward entry of water into the surface of the soil.

Infiltration Trench: A linear excavation, backfilled with gravel, used to filter pollutants and infiltrate storm water.

Integrated Pest Management Plan (IPMP): A balanced approach to pest management which incorporates the many aspects of plant health care in ways that mitigate harmful environmental impacts and protect human health.

Inlet: An entrance into a ditch, storm sewer, or other waterway.

Legacy Pollutants: Pollutants that are no longer in production but remain in site soils and groundwater and still have the potential to cause ecological and water quality impacts.

Material Storage Areas: On site locations where raw materials, products, final products, by-products, or waste materials are stored.

Maximum Extent Practicable (MEP): The technology-based permit requirement established by Congress in CWA section 402(p)(3)(B)(iii) that municipal dischargers of stormwater must meet. Technology-based requirements, including MEP, establish a level of pollutant control that is derived from available technology or other controls. MEP requires municipal dischargers to perform at maximum level that is practicable. Compliance with MEP may be achieved by emphasizing pollution prevention and source control BMPs in combination with structural and treatment methods where appropriate. The MEP approach is an ever evolving and advancing concept, which considers technical and economic feasibility.

Municipal Separate Storm Sewer System (MS4) Permit: : A NPDES permit issued by the Regional Water Quality Control Board for the discharge of Stormwater from Municipal Separate Storm Sewer Systems.

New Development: Land disturbing activities; structural development, including construction or installation of a building or structure, creation and replacement of impervious surfaces; and land subdivision.

Non-Stormwater Discharge: Any discharge to municipal separate storm drain that is not composed entirely of stormwater. Discharges containing process wastewater, non-contact cooling water, or sanitary wastewater are non-stormwater discharges.

Non-Structural Source Control Measure: Low technology, low cost activities, procedures or management practices designed to prevent pollutants associated with site functions and activities from being discharged with Stormwater runoff. Examples include good housekeeping practices, employee training, standard operating practices, inventory control measures, etc.

Notice of Intent (NOI): A formal notice to State Water Resources Control Board submitted by the owner/developer that a construction project is about to begin. The NOI provides information on the owner, location, type of project, and certifies that the permittee will comply with the conditions of the construction general permit.

NPDES Permit: An authorization, license, or equivalent control document issued by EPA or an approved State agency to implement the requirements of the NPDES program.

Operations and Maintenance (O&M): The continuing activities required to keep storm water management facilities and their components functioning in accordance with design objectives.

Outfall: The point where stormwater discharges from a pipe, channel, ditch, or other conveyance to a waterway.

Parking Lot: Land area or facility for the temporary parking or storage of motor vehicles used personally, for business or for commerce with an impervious surface area of 5,000 square feet or more, or with 25 or more parking spaces.

Permeability: A property of soil that enables water or air to move through it. Usually expressed in inches/hour or inches/day.

Pervious Surface/Area: A surface or area with a surface (i.e., soil, loose rock, permeable pavement, etc.) that allows water to infiltrate (soak) into the ground.

Planter Box: A structural facility filled with topsoil and gravel and planted with vegetation. The planter is completely sealed, and a perforated collection pipe is placed under the soil and gravel, along with an overflow provision, and directed to an acceptable destination point. The storm water planter receives runoff from impervious surfaces, which is filtered and retained for a period of time.

Pollutant: An elemental or physical material that can be mobilized or dissolved by water or air and creates a negative impact to human health and/ or the environment. Pollutants include suspended solids (sediment), heavy metals (such as lead, copper, zinc, and cadmium), nutrients (such as nitrogen and phosphorus), bacteria and viruses, organics (such as oil, grease, hydrocarbons, pesticides, and fertilizers), floatable debris, and increased temperature.

Pollutants of Concern: constituents that have exceeded Basin Plan Objectives, and California Toxics Rule chronic or acute objectives during monitoring at mass emission, receiving water, and land use stations.

Pollution Reduction: The practice of filtering, retaining, or detaining surface water runoff during and after a storm event for the purpose of maintaining or improving surface and/or groundwater quality.

Precipitation: Any form of rain or snow.

Predevelopment: The existing land use condition prior to the proposed development activity.

Practicable: Available and capable of being done, after taking into consideration existing technology, legal issues, and logistics in light of overall project purpose.

Pre-developed Condition: the native vegetation and soils that existed at a site prior to first development. The pre-developed condition may be assumed to be the typical vegetation, soil, and stormwater runoff characteristics of open space areas in coastal Southern California unless reasonable historic information is provided that the area was atypical.

Pre-project Condition: the condition of the site at the time of the proposed project.

Pretreatment: Treatment of wastewater before it is discharged to a wastewater collection system.

Process Wastewater: Wastewater that has been used in one or more industrial processes.

Project: development, redevelopment, and land disturbing activities. The term is not limited to "project" as defined under CEQA (Reference: California Public Resources Code § 21065).

Public Facility: A street, right-of-way, park, sewer, drainage, storm water management, or other facility that is either currently owned by the City/County or will be conveyed to the City/County for maintenance responsibility after construction.

Rainwater Harvesting: Rainwater harvesting is a BMP that stores and uses rainwater or stormwater runoff. This is consistent with the use of the term "reuse" contained in Order R4-2010-0108.

Receiving Stream: (for purposes of this Manual only) any natural or man-made surface water body that receives and conveys stormwater runoff.

Redevelopment: Land-disturbing activity that results in the creation, addition, or replacement of 5,000 square feet or more of impervious surface area on an already developed site. Redevelopment includes, but is not limited to: the expansion of a building footprint; addition or replacement of a structure; replacement of impervious surface area that is not part of a routine maintenance activity; and land disturbing activities related to structural or impervious surfaces. It does not include routine maintenance to maintain original line and grade, hydraulic capacity, or original purpose of facility, nor does it include emergency construction activities required to immediately protect public health and safety. Note: redevelopment as defined here is not the same as a "Redevelopment Project" as defined by California redevelopment law.

Redevelopment Project Area Master Plan (RPAMP): A plan submitted to the Regional Water Board for approval by a Permittee or a coalition of Permittees to establish standards for redevelopment projects within Redevelopment Project Areas, in consideration of exceptional site constraints that inhibit site-by-site or project-by-project implementation of post-construction requirements. See Section 4.E.IV.3 of Order R4-2010-0108.

Restaurant: A stand-alone facility that sells prepared foods and/or drinks for consumption, including stationary lunch counters and refreshment stands selling prepared foods and/or drinks for immediate consumption (SIC code 5812).

Retail Gasoline Outlet: Any facility engaged in selling gasoline and lubricating oils.

Retention Facility: A facility designed to receive and hold stormwater runoff. Rather than storing and releasing the entire runoff volume, retention facilities permanently retain a portion of the water on-site, where it infiltrates, evaporates, or is absorbed by surrounding vegetation. In this way, the full volume of storm water that enters the facility is not released off-site.

Retrofit: Retrofit projects implement structural treatment BMPs as a stand-alone project, without other site improvements. The BMP sizing requirements of this Technical Guidance Manual do not apply to retrofit projects.

Runoff: Water originating from rainfall and other precipitations (e.g., sprinkler irrigation) that is found in drainage facilities, rivers, streams, springs, seeps, ponds, lakes, wetlands, and shallow groundwater.

Runon: Stormwater surface flow or other surface flow which enters property other than that where it originated.

Secondary Containment: Structures, usually dikes or berms, surrounding tanks or other storage containers and designed to catch spilled material from the storage containers.

Sedimentation: The process of depositing soil particles, clays, sands, or other sediments that were picked up by runoff.

Sediments: Soil, sand, and minerals washed from land into water usually after rain, that accumulate in reservoirs, rivers, and harbors, destroying aquatic animal habitat and clouding the water so that adequate sunlight might not reach aquatic plants.

Site: land or water area where any "facility" or "activity" is physically located or conducted including adjacent land used in connection with the facility or activity.

Source Control BMP or Measure: Any schedules of activities, structural devices, prohibitions of practices, maintenance procedures, managerial practices or operational practices that aim to prevent Stormwater pollution by reducing the potential for contamination at the source of pollution.

Source Control BMPs: Operational practices or design features that prevent pollution by reducing potential pollutants at the source.

Spill Guard: A device used to prevent spills of liquid materials from storage containers.

Spill Prevention Control and Countermeasures Plan (SPCC): Plan consisting of structures, such as curbing, and action plans to prevent and respond to spills of hazardous substances as defined in the Clean Water Act.

Storm Drains: Above and below ground structures for transporting stormwater to streams or outfalls for flood control purposes.

Storm Drain System: Network of above and below-ground structures for transporting stormwater to streams or outfalls.

Storm Event: A rainfall event that produces more than 0.1 inch of precipitation and is separated from the previous storm event by at least 72 hours of dry weather.

Stormwater Discharge Associated with Industrial Activity: Discharge from any conveyance which is used for collecting and conveying stormwater which is related to manufacturing processing or raw materials storage areas at an industrial plant [see 40 CFR 122.26(b)(14)].

Stormwater: Stormwater runoff, snow-melt runoff, surface runoff, and drainage, excluding infiltration and irrigation tailwater.

Structural BMP or Control Measure: Any structural facility designed and constructed to mitigate the adverse impacts of stormwater and urban runoff pollution (e.g. canopy, structural enclosure). The category may include both Treatment Control BMPs and Source Control BMPs.

Total Project Area: Total project area (or “gross project area”) for new development and redevelopment projects is the disturbed, developed, and undisturbed portions within the project’s property (or properties) boundary, at the project scale submitted for first approval. Areas proposed to be permanently dedicated for open space purposes as part of the project are explicitly included in the "total project area." Areas of land precluded from development through a restrictive covenant, conservation easement, or other recorded document for the permanent preservation of open space prior to project submittal shall not be included in the "total project area."

Total Suspended Solids (TSS): Matter suspended in stormwater excluding litter, debris, and other gross solids exceeding 1 millimeter in diameter.

Treatment Control BMP or Measure: Any engineered system designed to remove pollutants by simple gravity settling of particulate pollutants, filtration, biological uptake, media adsorption or any other physical, biological, or chemical process.

Treatment: The application of engineered systems that use physical, chemical, or biological processes to remove pollutants. Such processes include, but are not limited to, filtration, gravity settling, media adsorption, biodegradation, biological uptake, chemical oxidation and UV radiation.

Tributary Area: The area from which all runoff produced flows to the same specific discharge point.

Vegetated Facilities: Stormwater management facilities that rely on plantings to enhance their performance. Plantings can provide wildlife habitat and enhance many facility functions, including infiltration, pollutant removal, water cooling, flow calming, and prevention of erosion.

Vegetated Swale: A long and narrow, trapezoidal or semicircular channel, planted with a variety of trees, shrubs, and grasses or with a dense mix of grasses. Stormwater runoff from impervious surfaces is directed through the swale, where it is slowed and in some cases infiltrated, allowing pollutants to settle out. Check dams are often used to create small ponded areas to facilitate infiltration.

APPENDIX B : MAPS

NOTES:

1. Contact the local permitting authority for more detailed maps.
2. Existing Urban Area maps are current as of 11/2/10.

Legend

River		Santa Paula	
Lake		Simi Valley	
National Forest		Thousand Oaks	
10-Digit Hydrologic Unit Boundary		Port Hueneme	

Existing Urban Area

Camarillo	Ventura
Fillmore	Ojai
Moorpark	Urban County
Oxnard	Non-Urban County
	Adjacent County

10 5 0 10 Miles

Hydrologic Areas

Ventura County, CA

Geosyntec consultants

Figure **B-1**

Oakland Office | April 2010

P:\GIS\Ventura_County_Manual\Projects\B-1_Ventura_County_Watersheds_MS4_Urban_Areas_042910.mxd, WHL, September 1, 2010

Legend

- BIOL Designated Waterbody
- 303(d) Listed Waterbody
- Environmentally Sensitive Habitat Areas
- Lake
- National Forest

Existing Urban Area

- Camarillo
- Fillmore
- Moorpark
- Oxnard
- Santa Paula
- Simi Valley
- Thousand Oaks
- Port Hueneme
- Ventura
- Ojai
- Urban County
- Non-Urban County
- Adjacent County

10 5 0 10 Miles

Environmentally Sensitive Areas

Ventura County, CA

Geosyntec consultants

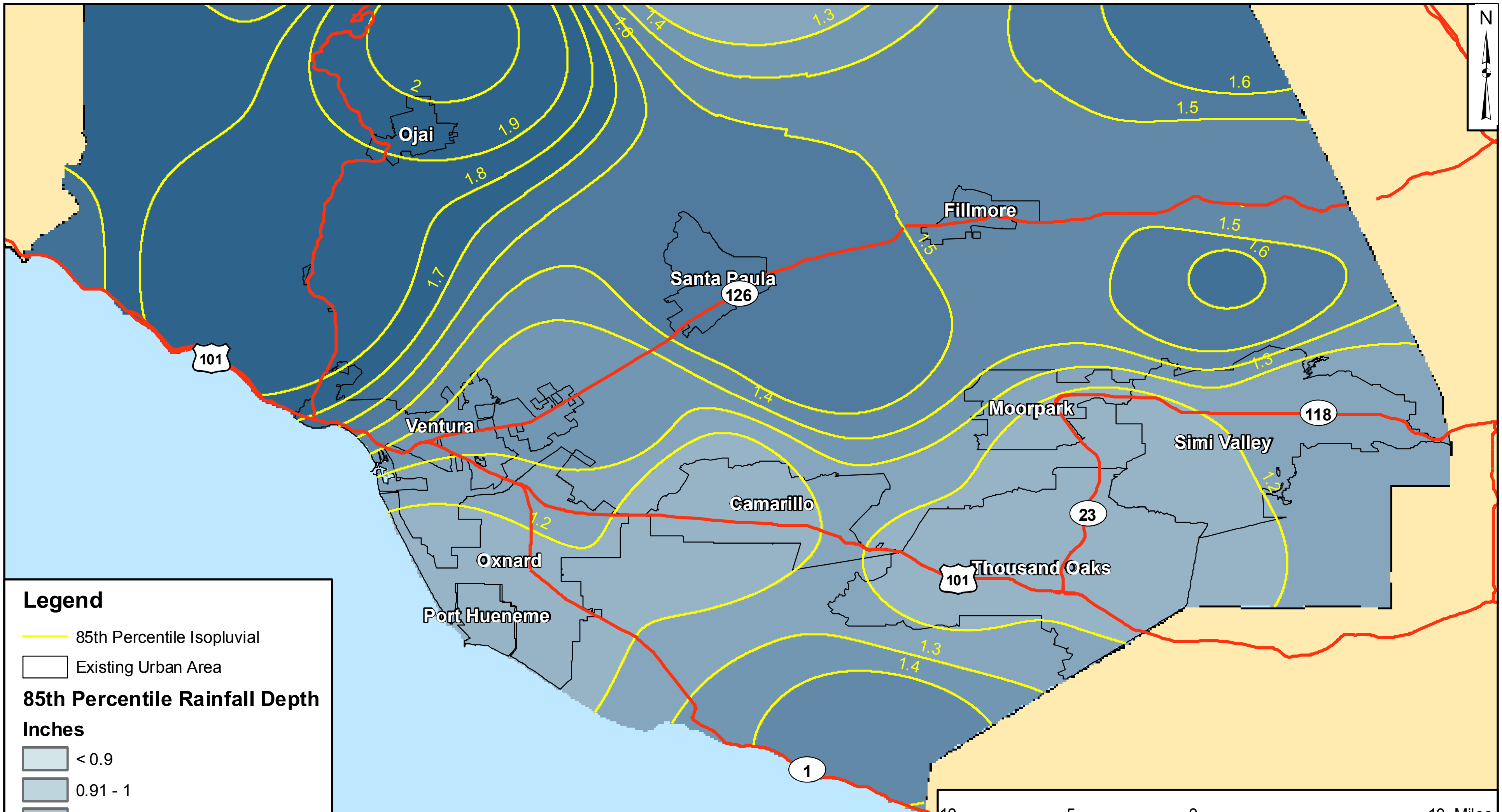
Figure

B-2

Oakland Office

April 2010

B-2_Ventura_County_ESA_042910.mxd, WHL, September 1, 2010



Legend

85th Percentile Isopluvial

Existing Urban Area

85th Percentile Rainfall Depth

Inches

< 0.9

0.91 - 1

1.01 - 1.1

1.11 - 1.2

1.21 - 1.3

1.31 - 1.4

1.41 - 1.5

1.51 - 1.6

1.61 - 1.7

1.71<

Adjacent County

Ventura County 85th Percentile Rain Depths

Ventura County, CA

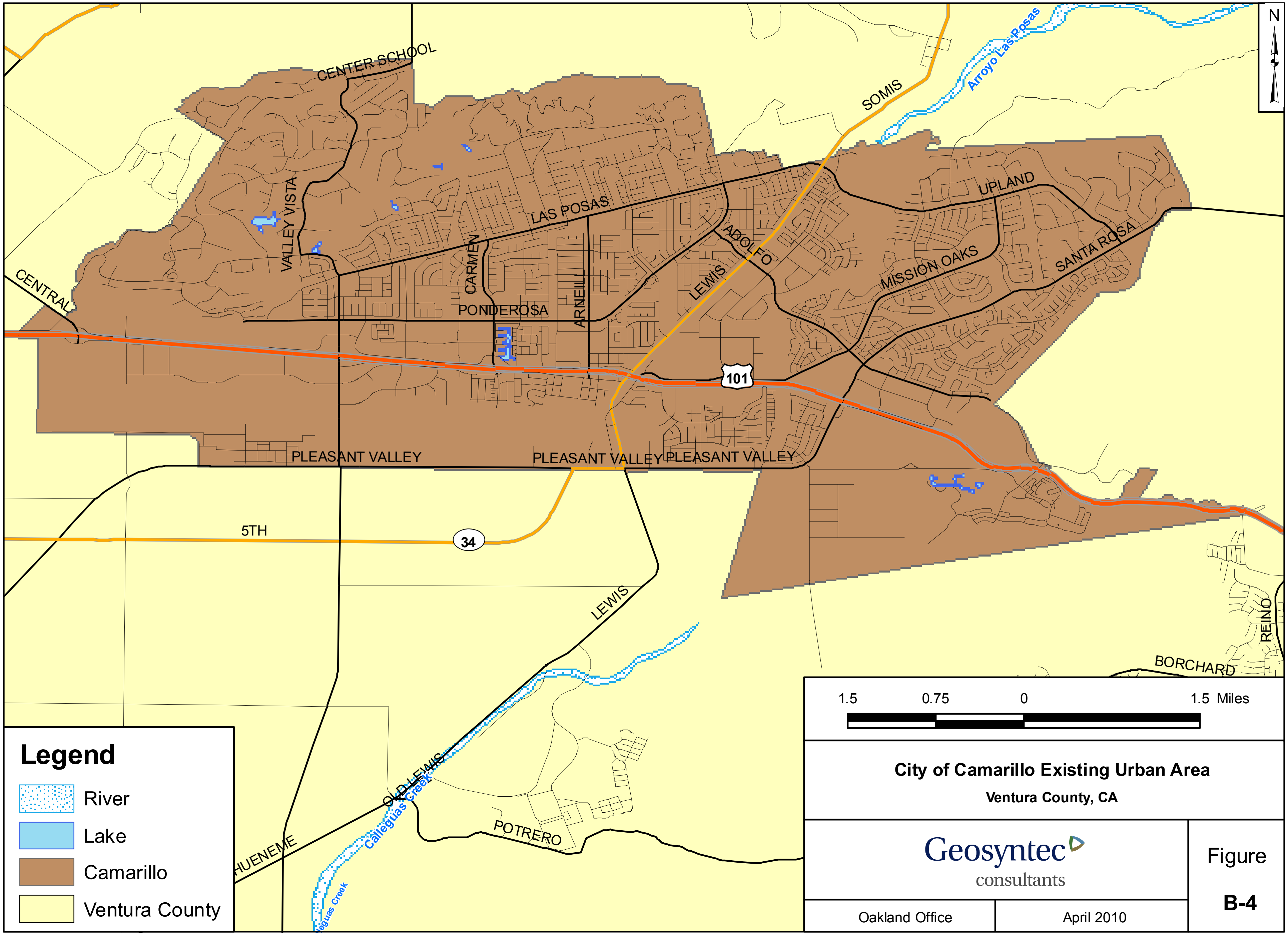
Geosyntec consultants

Figure

B-3

Oakland Office | September 2010

P:\GIS\Ventura_County_Manual\Projects\B-3_Ventura_County_85th_Isopluvials_092010.mxd, WHL, September 20, 2010



City of Camarillo Existing Urban Area

Ventura County, CA

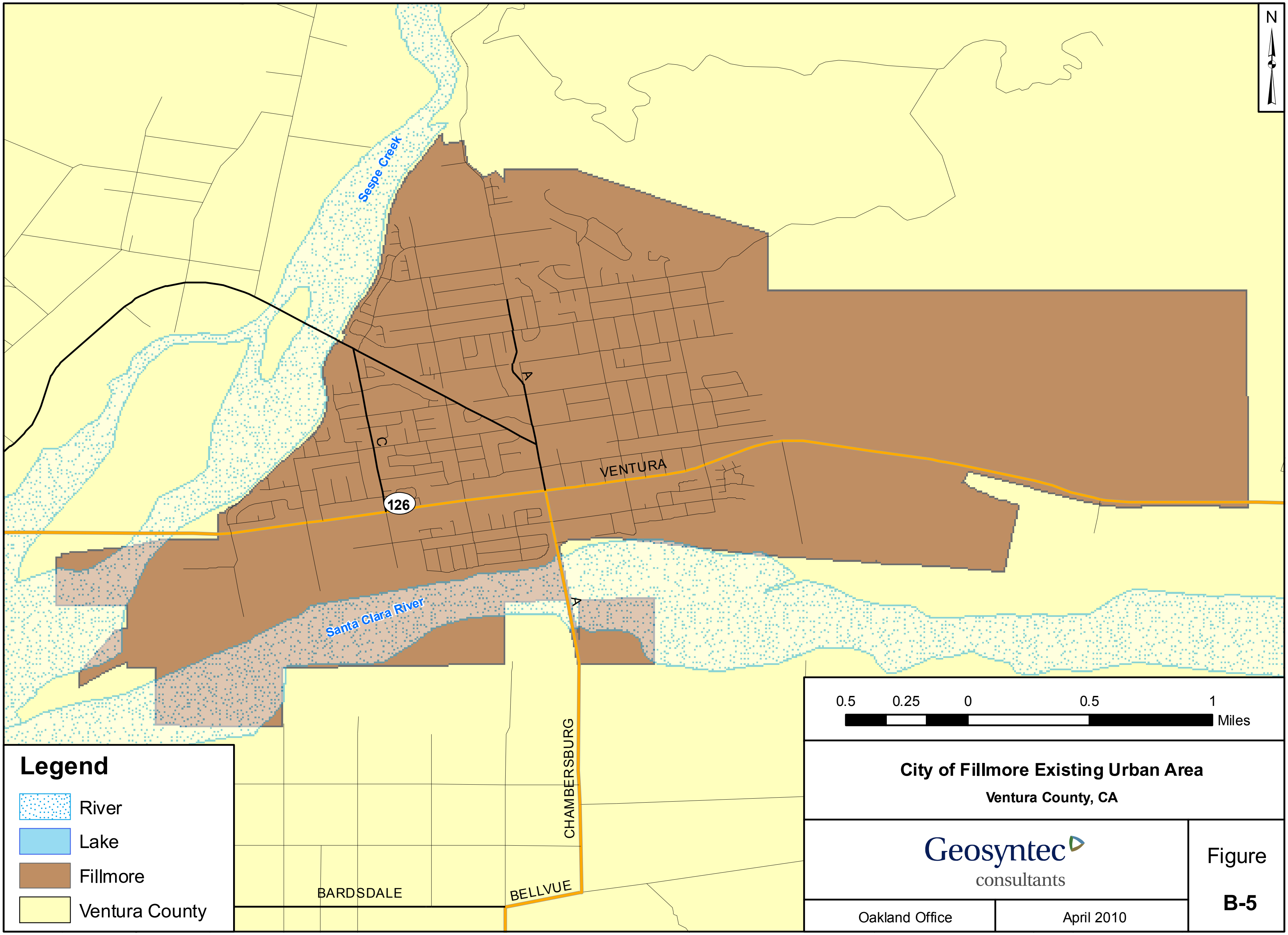
Geosyntec consultants

Oakland Office | April 2010

Figure

B-4

P:\GIS\Ventura_County_Manual\Projects\B-4_Camarillo_MS4_Pop_Areas_092410.mxd, WHL, September 24, 2010



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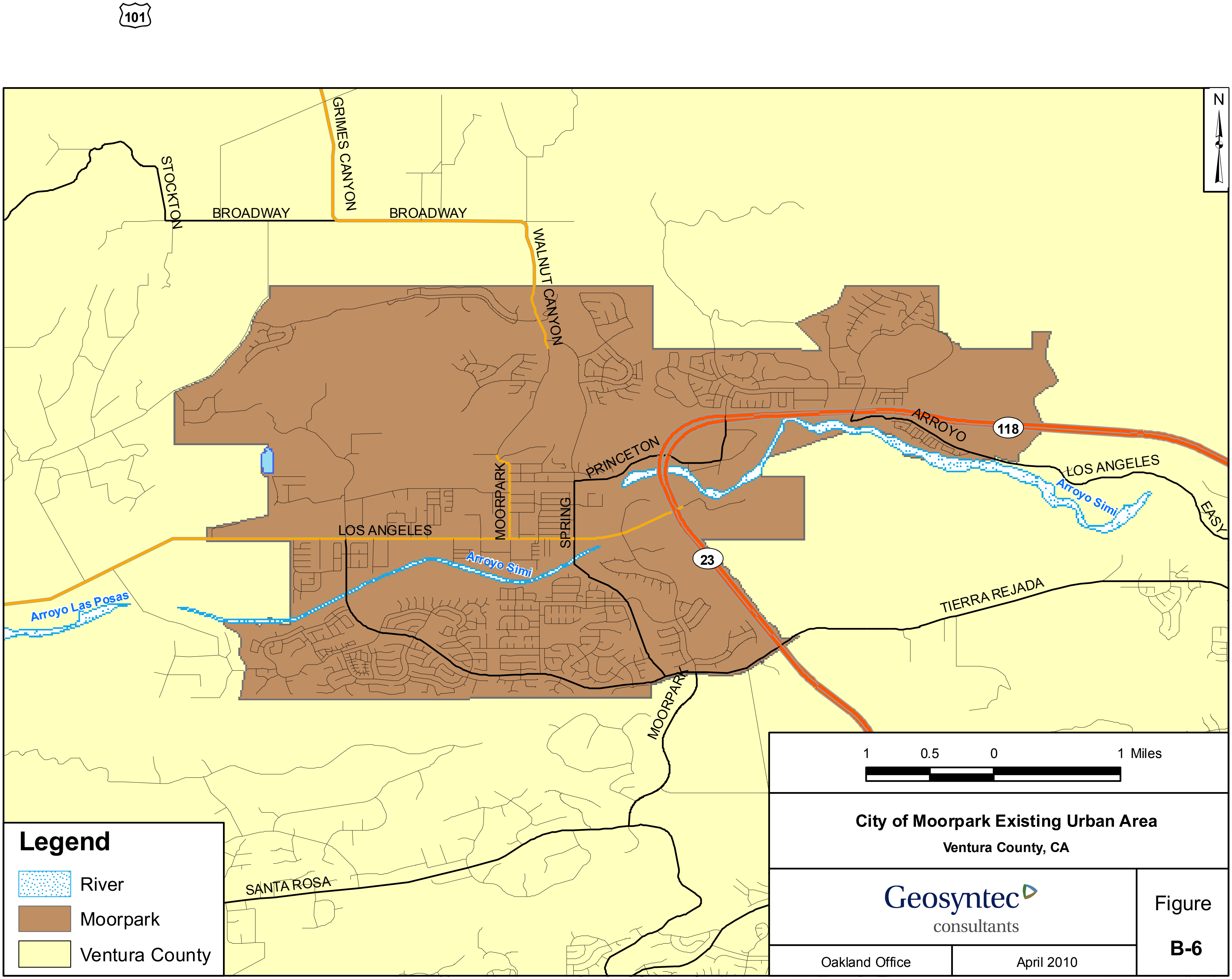

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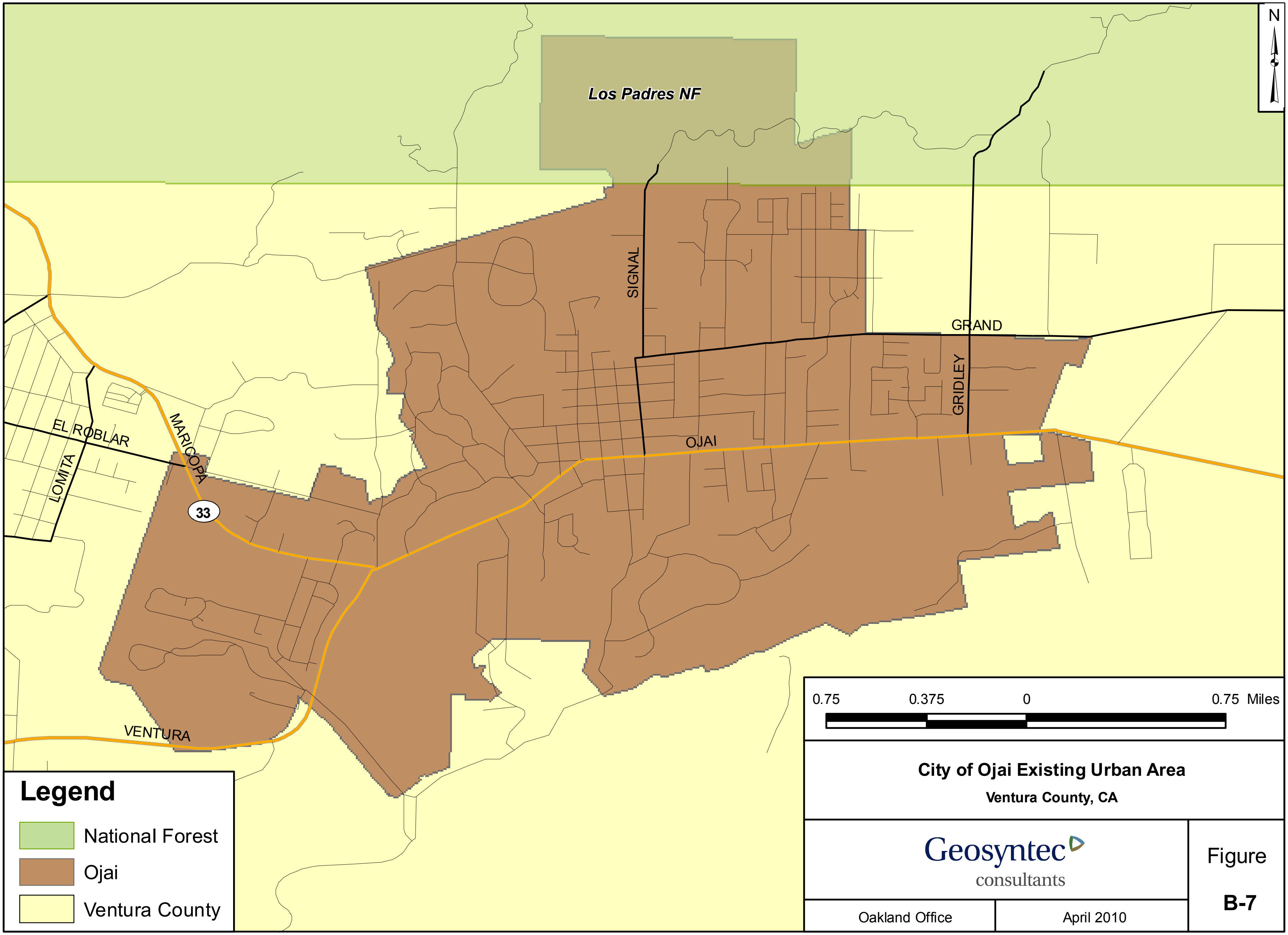


City of Ojai Existing Urban Area

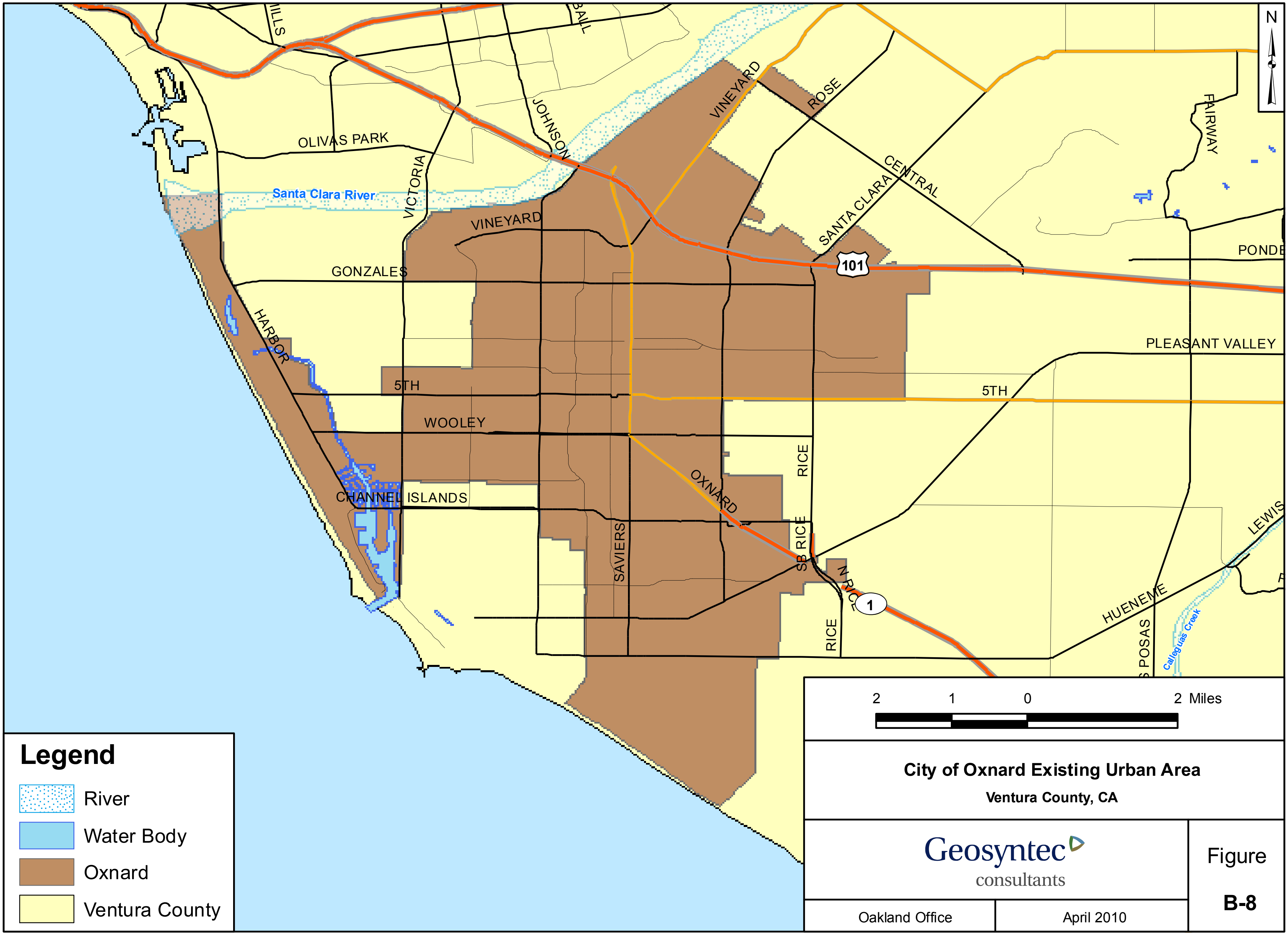
Ventura County, CA

Geosyntec consultants

Oakland Office | April 2010

Figure **B-7**

P:\GIS\Ventura_County_Manual\Projects\B-7_Ojai_MS4_Pop_Areas_092310.mxd, WHL, September 23, 2010



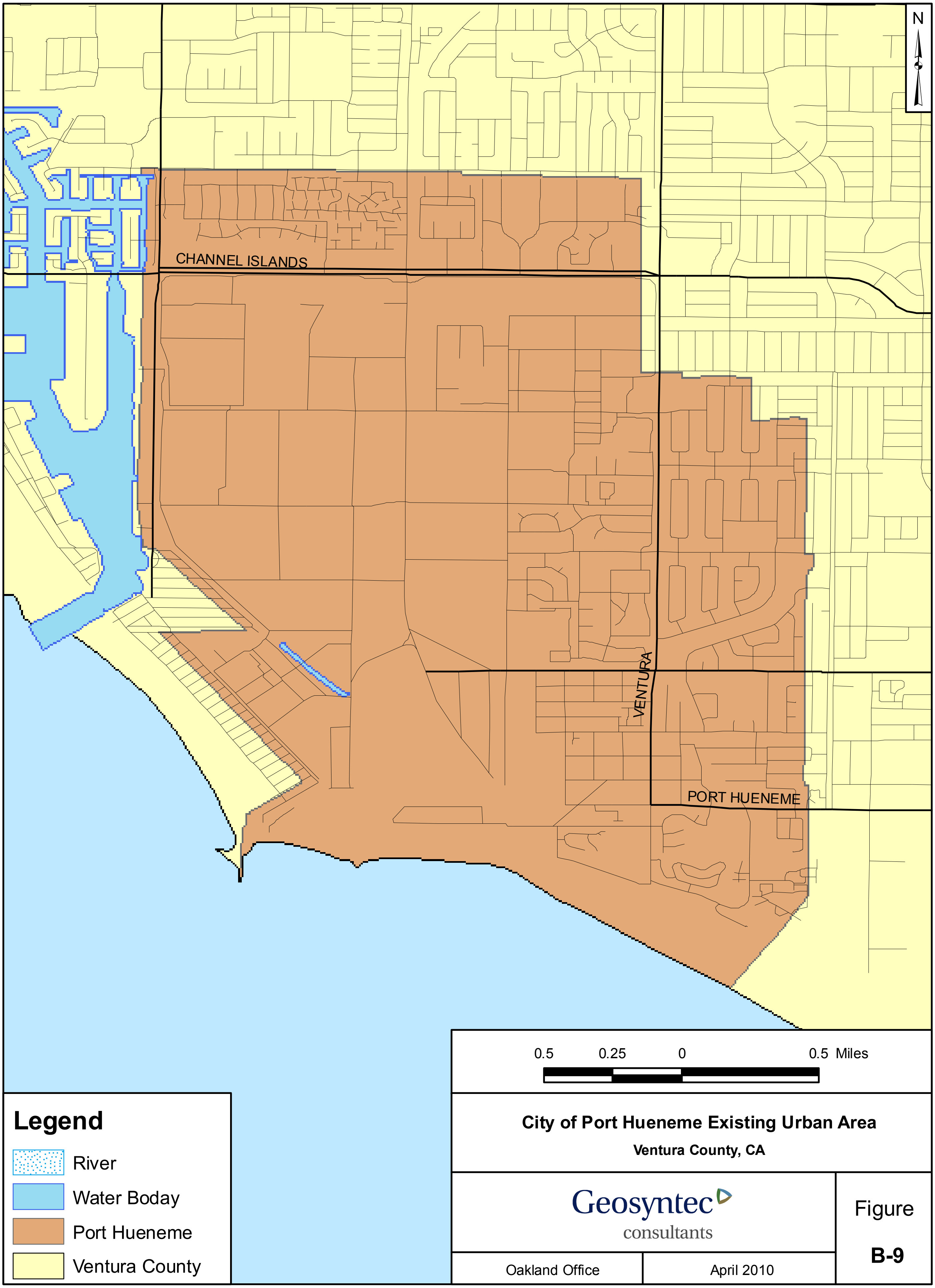
City of Oxnard Existing Urban Area

Ventura County, CA

Geosyntec consultants

Oakland Office	April 2010	Figure B-8

P:\GIS\Ventura_County_Manual\Projects\B-8_Oxnard_MS4_Pop_Areas_092310.mxd, WHL, September 23, 2010



City of Port Hueneme Existing Urban Area

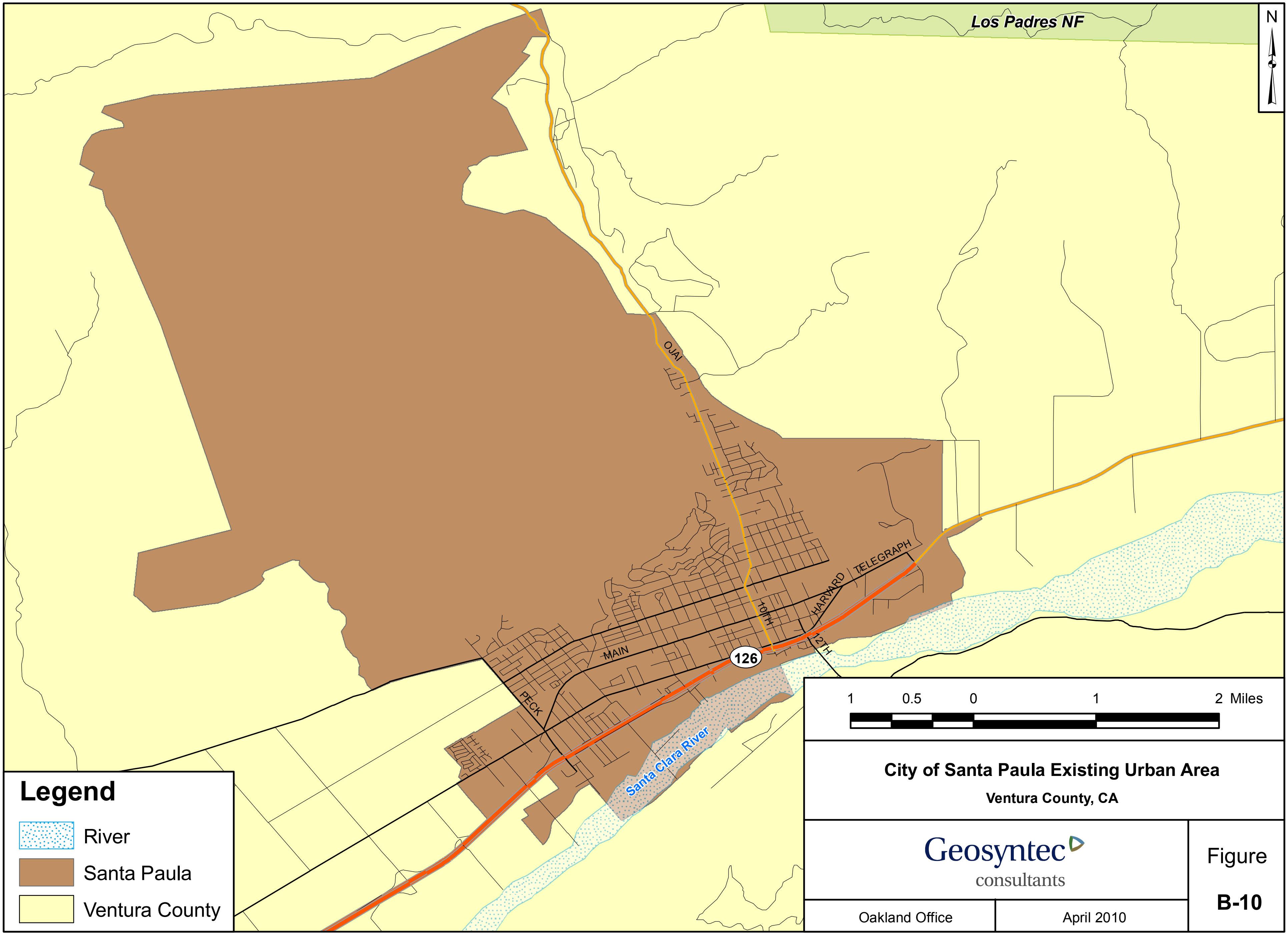
Ventura County, CA

Geosyntec consultants

Oakland Office | April 2010

Figure **B-9**

P:\GIS\Ventura_County_Manual\Projects\B-9_Port_Hueneme_MS4_Urban_Areas_092310.mxd, WHL, September 23, 2010



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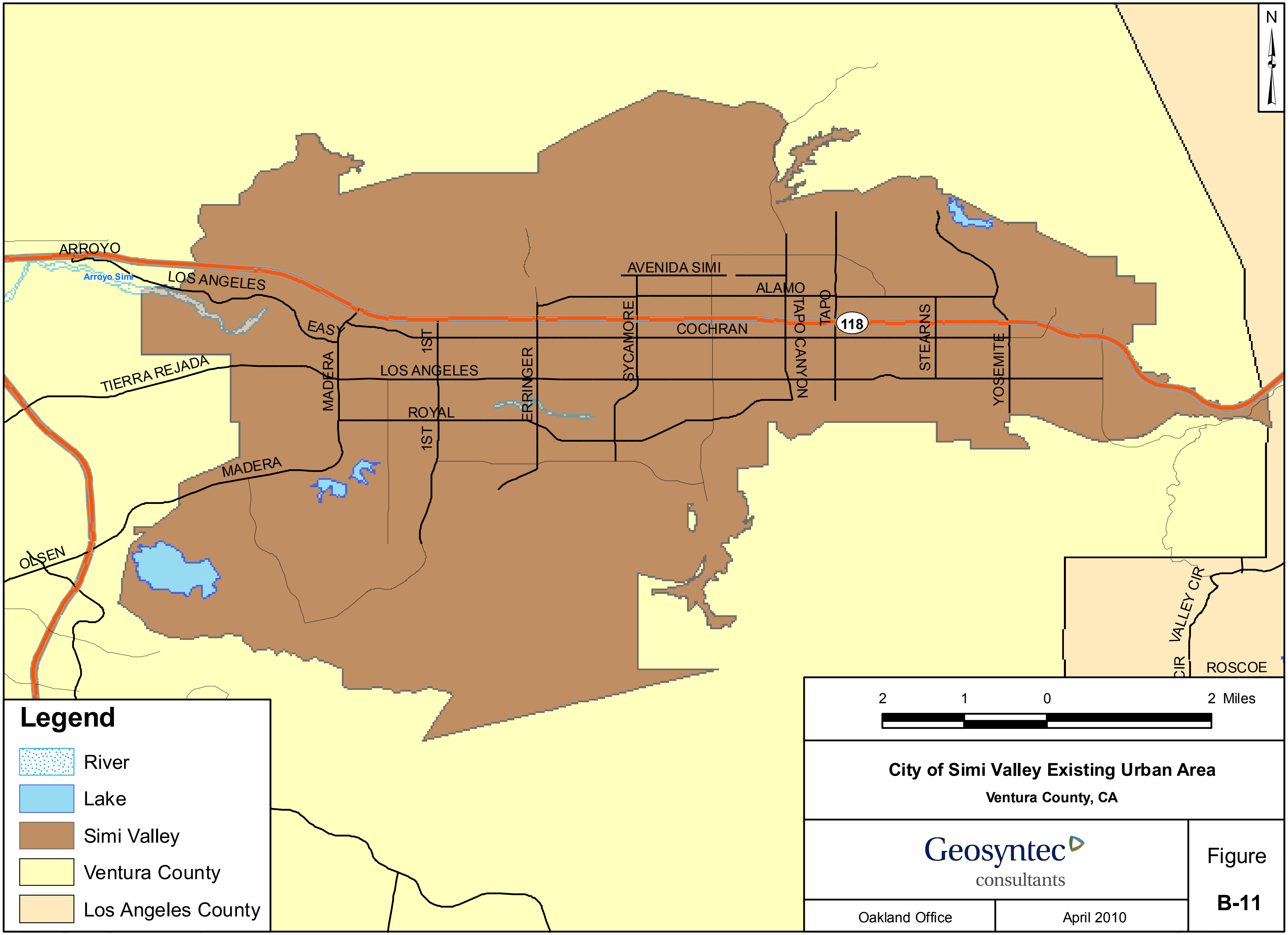
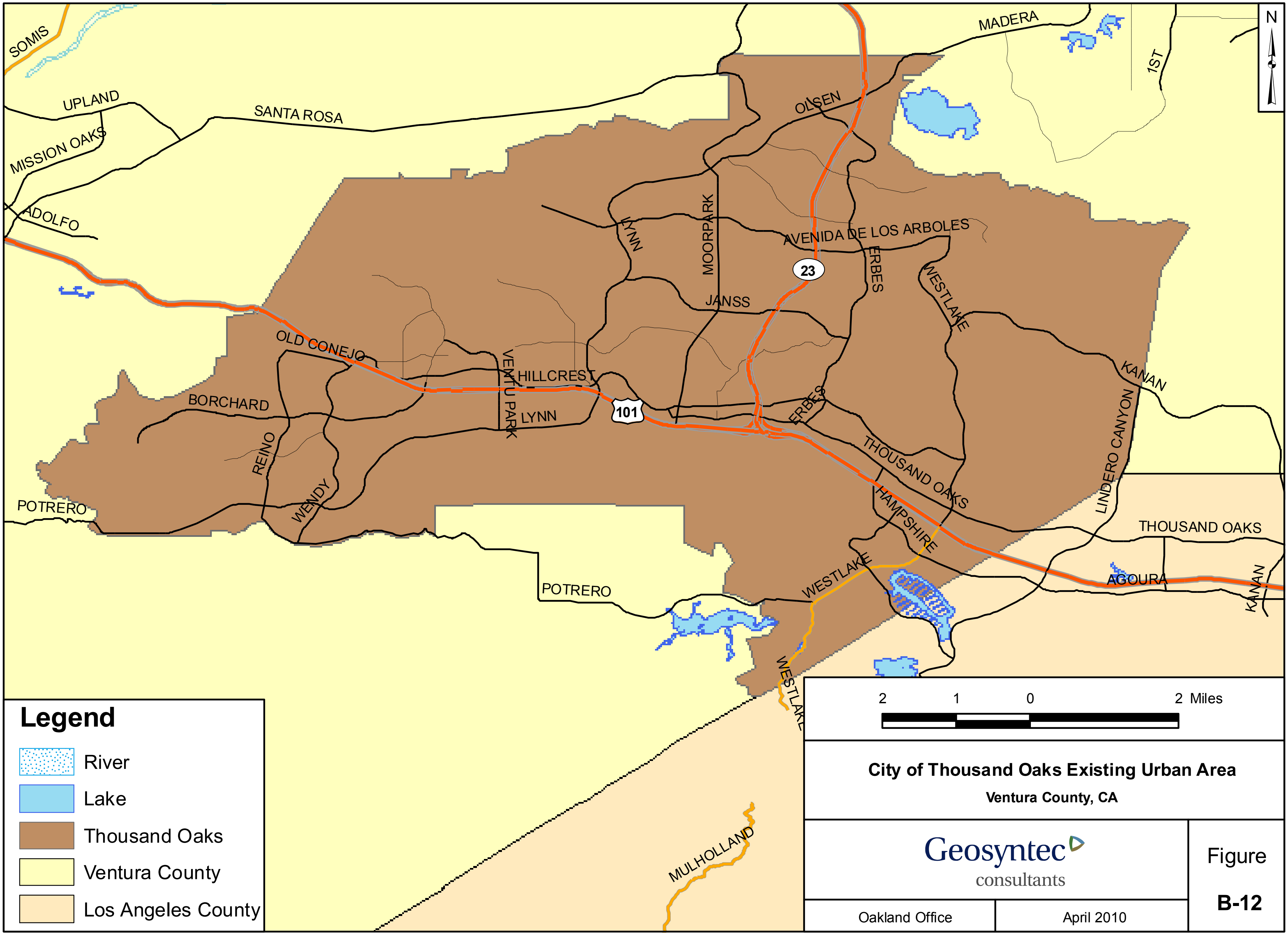


P:\GIS\Ventura_County_Manual\Projects\Simi_Valley_MS4_Pop_Areas_092310.mxd, WHL, September 23, 2010



City of Thousand Oaks Existing Urban Area

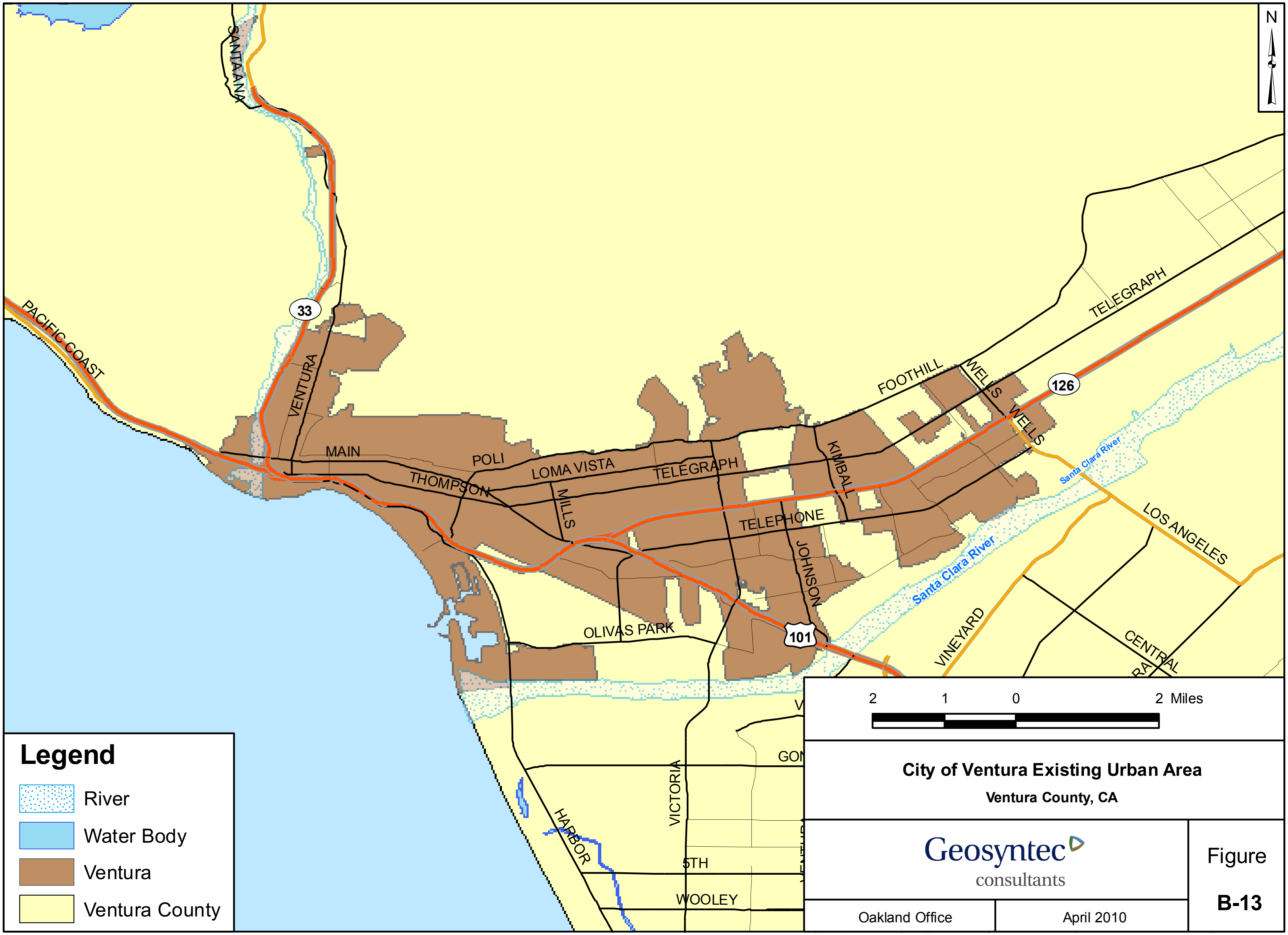
Ventura County, CA

Geosyntec consultants

Oakland Office | April 2010

Figure **B-12**

P:\GIS\Ventura_County_Manual\Projects\B-12_Thousand_Oaks_MS4_Pop_Areas_092310.mxd, WHL, September 23, 2010



City of Ventura Existing Urban Area

Ventura County, CA

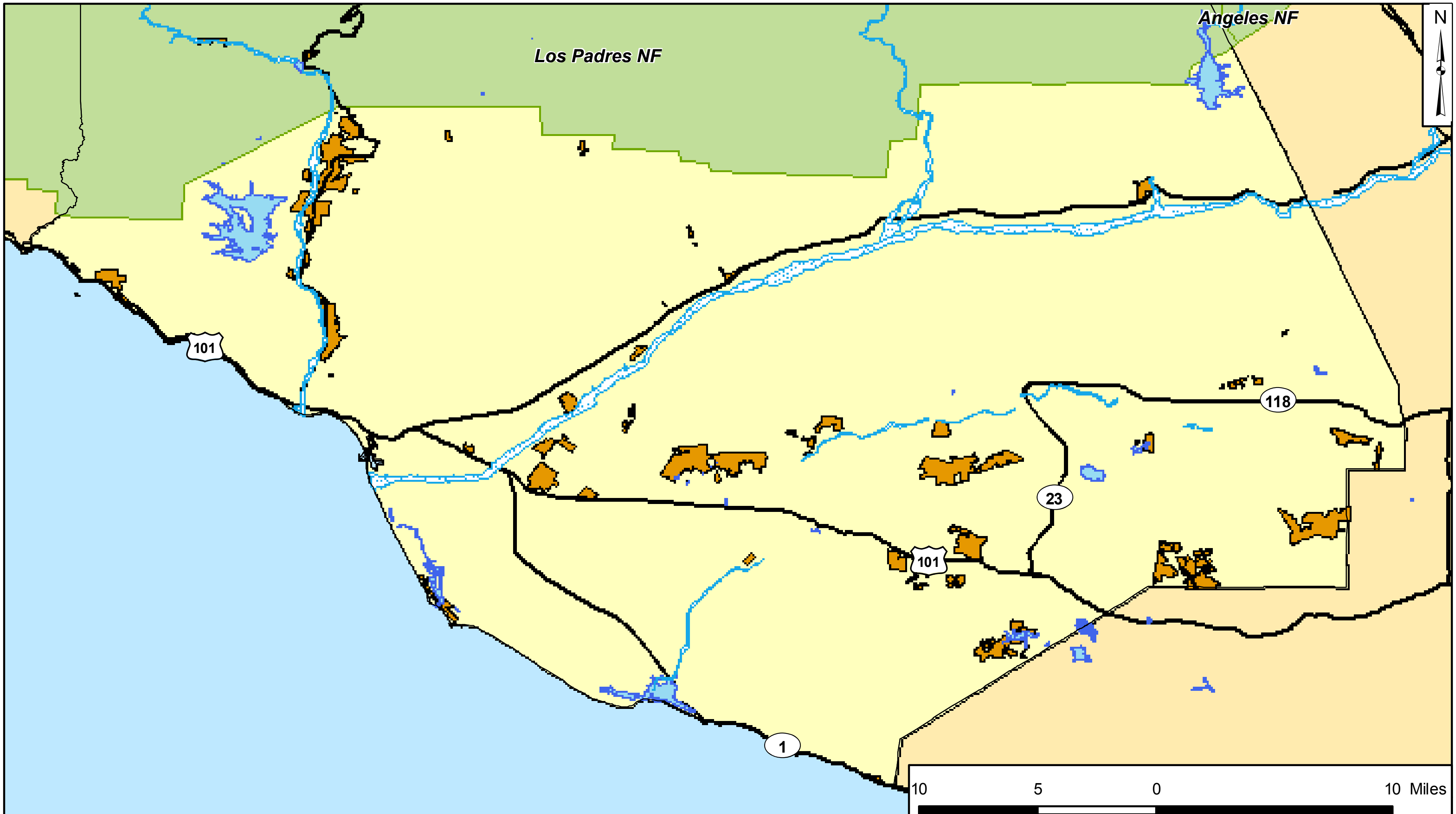
Geosyntec consultants

Oakland Office | April 2010

Figure

B-13

P:\GIS\Ventura_County_Manual\Projects\B-13_Ventura_MS4_Pop_Areas_092310.mxd, WHL, September 23, 2010



Legend

- River
- Lake
- Unincorporated Urban County
- National Forest
- Non-Urban County
- Adjacent County

Note: An Unincorporated Urban Center is an existing or planned community which is located in an Area of Interest where no city exists. The unincorporated urban center represents the focal center for community and planning activities within an Area of Interest. For example, the Community of Piru represents the focal center in the Piru Area of Interest. This map represents the existing Unincorporated Urban Centers as defined by the Ventura County General Plan.

Ventura County Unincorporated Urban Areas

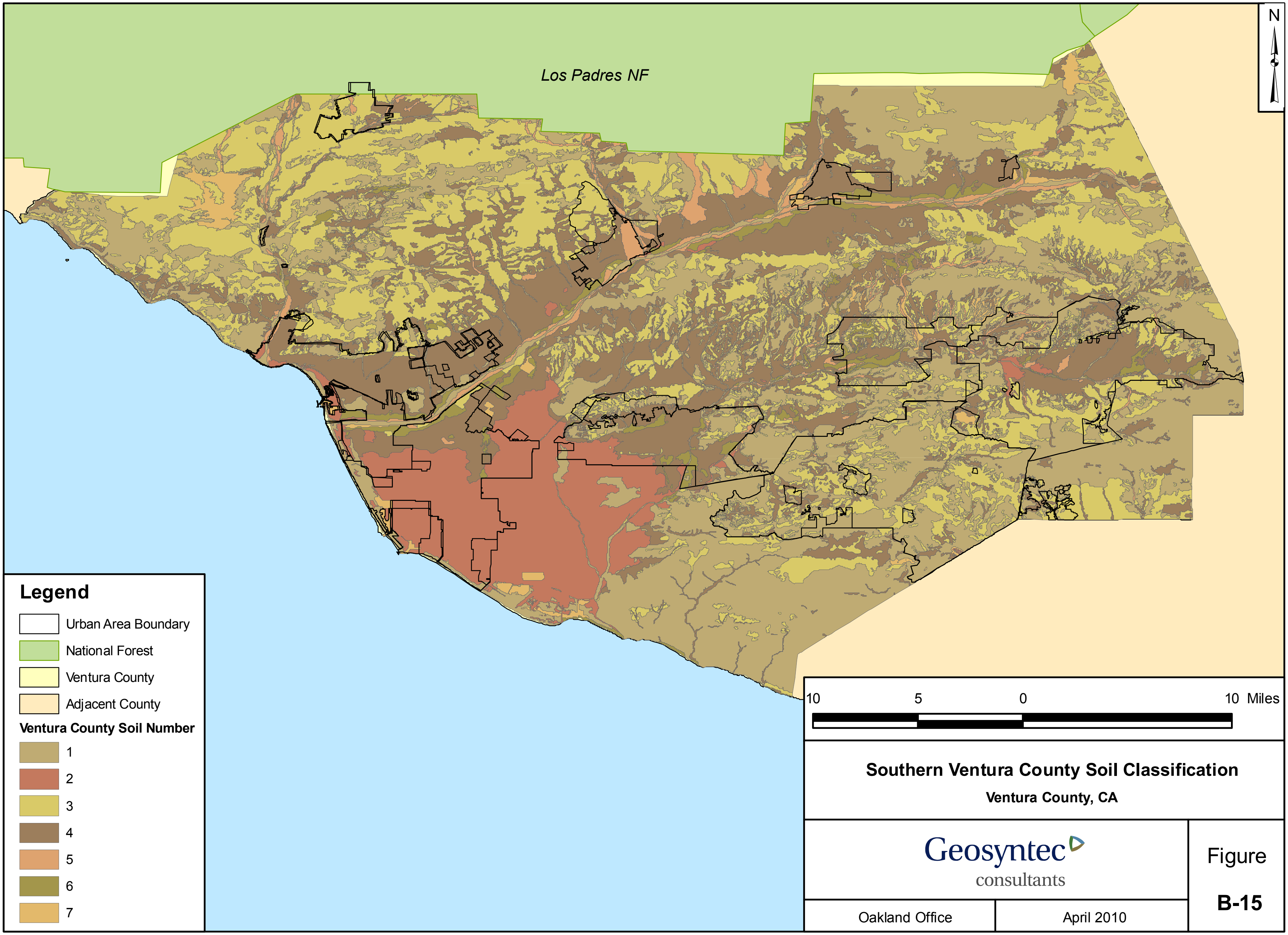
Ventura County, CA

Geosyntec consultants

Figure **B-14**

Oakland Office	April 2010

P:\GIS\Ventura_County_Manual\Projects\B-14_Ventura_County_Urban_092310.mxd, WHL, September 23, 2010



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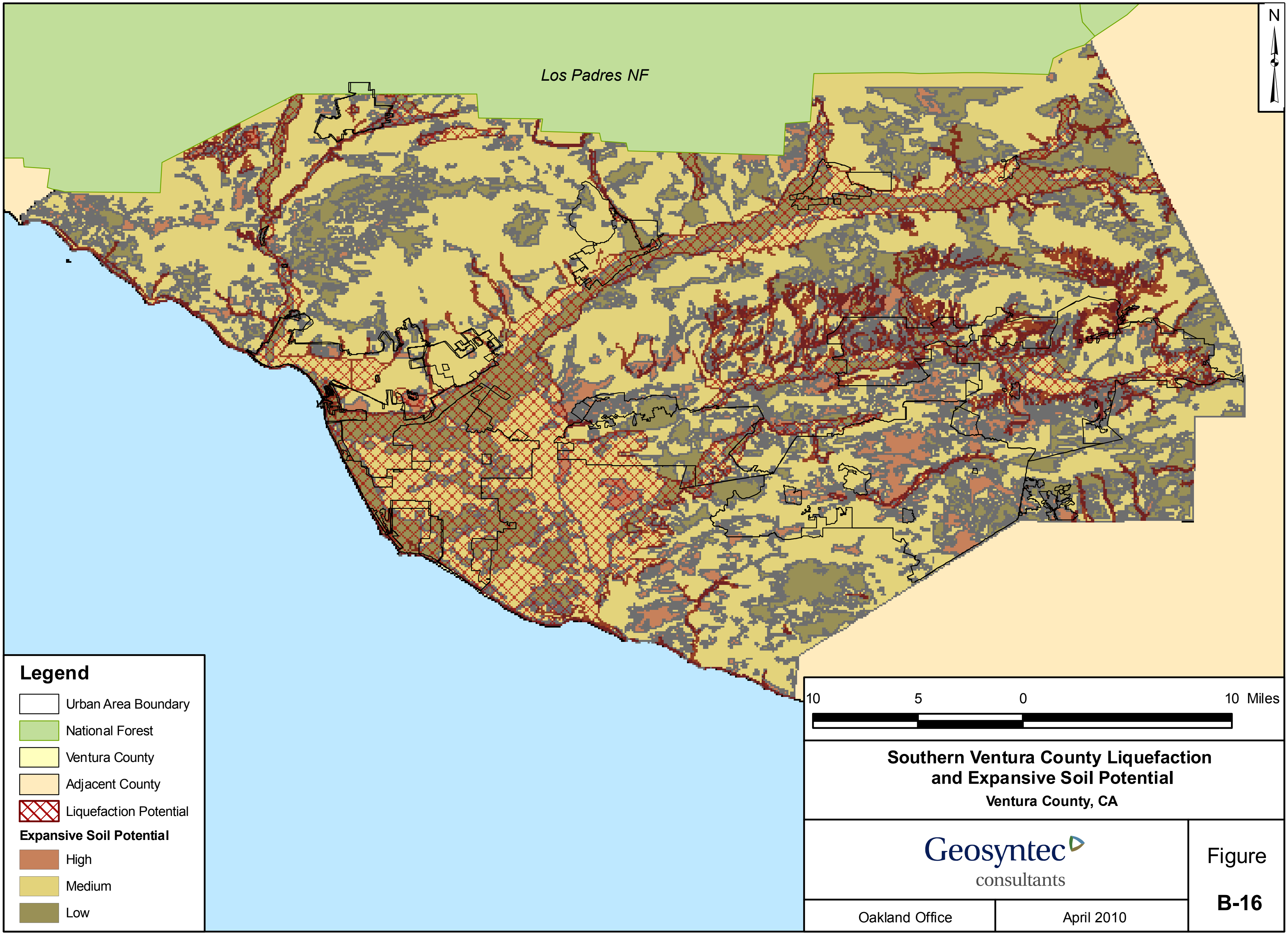

P:\GIS\Ventura_County_Manual\Projects\B-16_Liq_Expan_Soils_092310.mxd, WHL, September 23, 2010

APPENDIX C : SITE SOIL TYPE AND INFILTRATION TESTING

C.1 Introduction

The purpose of site soil and infiltration testing is to more accurately determine where LID and structural treatment BMPs should be located and if infiltration is feasible on the site. The preliminary site assessment, discussed in Section 3, will likely reduce the number of test pit investigations needed by identifying candidate test sites that are most amenable to infiltration. This section summarizes the methods for conducting (1) soil test pit investigations and (2) infiltration testing at key locations identified in the preliminary site assessment that require further investigation.

A qualified soil scientist or geotechnical professional should conduct the test pit investigation and infiltration tests. The professional should be experienced with the testing procedures as well as the hydraulic functioning of the potential BMPs to ensure that additional information regarding BMP siting is acquired during the test pit investigation and infiltration tests.

This appendix is not intended to be applied as a protocol for conducting soil and infiltration testing. Instead, this section is provided to assist in specifying and standardizing soil and infiltration testing techniques across sites within Ventura County where development is occurring.

C.2 Test Pit Investigations

A test pit investigation is an integral part of assessing site soil conditions. Soil maps and hydrologic soil groups are based on regional data and provide only a general understanding of what to expect; however, there are undoubtedly unknowns that will be discovered during these initial field observations. A test pit investigation involves digging or excavating a test pit (deep hole). By excavating a test pit, overall soil conditions (both vertically and horizontally) can be observed in addition to the soil horizons. To maximize the knowledge gained during the test pit investigation, many tests and observations should be conducted during this process.

Test pits should be excavated to a depth at least three feet deeper than the proposed bottom of non-infiltration BMPs and at least eleven feet deeper than the proposed bottom of infiltration BMPs. A project that imports fill must characterize the proposed soil profile at the specified depths. For example, if the proposed depth of fill is 5 feet below grade and an infiltration BMP is to be used in the location of the fill, both the fill and the native subsoil require soil characterization. Figure C-1 illustrates the proposed soil profile that would result with 3 feet of fill. Since the test pit must be excavated to a depth that is 11 feet deeper than the bottom of the proposed infiltration BMP, a test pit investigation of the top 8 feet of native subsoil is required, in addition to the laboratory sample of the fill material. Characterization of the fill material should be conducted in a laboratory. It is recommended that soil compaction is limited in the location of a proposed infiltration BMP.

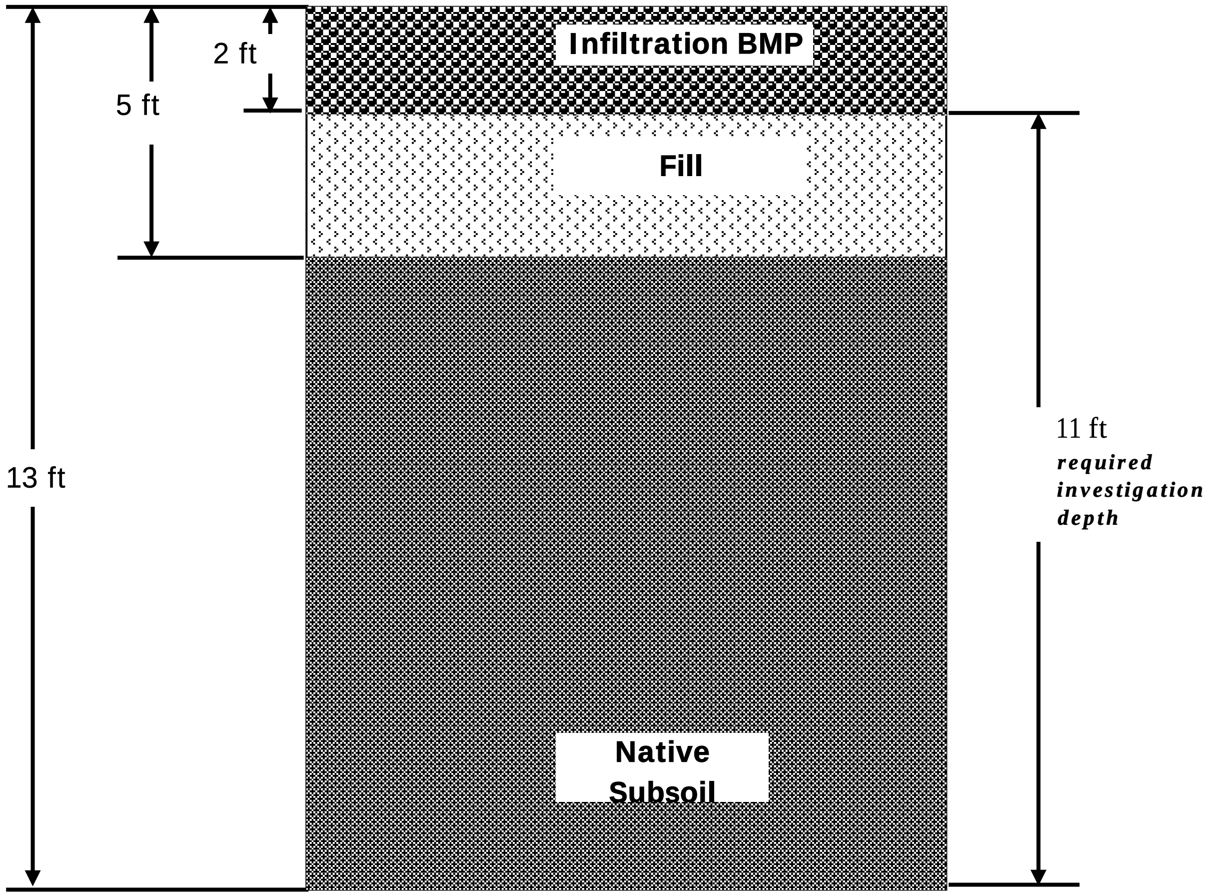

Figure C-1: Post-fill Soil Profile

As the test pit is excavated, the following measurements should be made:

Standard penetration testing to determined the relative density as it changes with depth (minimum intervals of 2 - 3 feet), and

Infiltration testing with at least one test occurring at the proposed bottom of the BMP and one test occurring of the bottom of the test pit (11 feet below the bottom of the infiltration BMP).

In addition, many observations should be made during and after the excavation of the soil pit, including:

- Elevation of groundwater table or indications of seasonally high groundwater table should be noted using the NRCS hydric soil field indicators guide (NRCS, 2003).
- Soil horizon observations, including: depths indicating upper and lower boundaries of the soil horizons, depths to limiting layers (i.e., bedrock and clay), soil textures, colors and their patterns, and estimates of the type and percent of coarse fragments.

- Locations and descriptions of macropores (i.e., pores and roots).
- Other pertinent information/observations.

The number of test pits required depends largely on the specific site and the proposed development plan. Additional tests should be conducted if local conditions indicate significant variability in soil types, geology, water table elevations, bedrock, topography, etc. Similarly, uniform site conditions may indicate that fewer test pits are required. Excessive testing and disturbance of the soil prior to construction is not recommended. When test pit investigations are complete, including infiltration testing, the pits should be refilled with the original soil and the surface replaced with the original topsoil.

C.3 Infiltration Testing

There are a variety of infiltration field test methodologies available to determine the infiltration rate of a soil. Infiltration tests should be conducted in the field in order to ensure that the measurements are representative of actual site conditions (including inherent heterogeneity). As mentioned above, usually infiltration rates should be determined at a minimum of two locations in each test pit and one must be conducted at the proposed bottom depth of the BMP. The actual number of infiltration tests required depends on the soil conditions; if the soils are highly variable, more tests may be required. To ensure groundwater is protected and that the infiltration BMP is not rendered ineffective by overload, it is important to periodically verify infiltration rates of the constructed BMP(s).

For BMPs that infiltrate water through the surface soil layer (e.g., bioretention areas, permeable pavement), choosing a method that measures infiltration in surface soils is important. For infiltration trenches and drywells, infiltration will occur at a greater depth in the soil matrix; therefore, borehole methods may be more appropriate.

Depending on the type of infiltration BMP and depth at which the infiltration test should be conducted, there are several types of infiltration tests that can be used including: disc permeameters, single and double ring infiltrometers, and borehole permeameters. Disc permeameters are typically used to provide estimates of soil near saturation but can prove to be difficult due to measures of three dimensional flow. This device is also commonly used for assessing infiltration rates of already constructed permeable pavements and is generally not used for assessing infiltration rates prior to site disturbance; therefore, the disc permeameter method will not be discussed further in this Appendix. Single and double ring infiltrometers directly measure vertical flow into the surface of the soil. Double ring infiltrometers account for lateral flow boundary affects with the addition of an outer water reservoir and are generally the preferred method for surface infiltration. Borehole permeameters are best suited to collect infiltration measurements below the soil surface. Two subsurface infiltration methods are discussed below including the Guelph and falling-head permeameters.

C.4 Double Ring Infiltrometer

The double ring infiltrometer method consists of driving two cylinders, one inside the other, into the ground and partially filling them with water and maintaining the liquid at a constant level (ASTM D3385-94). The volume of water added to the inner ring from a separate water reservoir, to maintain the constant head level is comparable to the volume of water infiltrating into the soil. The volume of water added to the inner ring divided by the time period for which the water was added is equal to the infiltration rate. A photograph of a common double ring infiltrometer is provided in Figure C-2.

Figure C-2: Double Ring Infiltrometer

Photo Credit: Geosyntec Consultants (Braga and Fitsik, 2008)

C.5 Borehole Guelph Infiltration Test

For shallow boreholes, the Guelph Permeameter has been developed as a field portable kit. This permeameter consists of a tube that is placed in a hand-drilled shallow borehole and water is provided to the tube through a separate reservoir. Water loss in the reservoir is used to estimate the hydraulic conductivity of the soil, which may be used to calculate infiltration based on various standard models (Soil Moisture Equipment, 2005). A photograph of a Guelph Permeameter is provided in Figure C-3. It is important to remember that this method will include vertical and lateral water flow from the borehole.

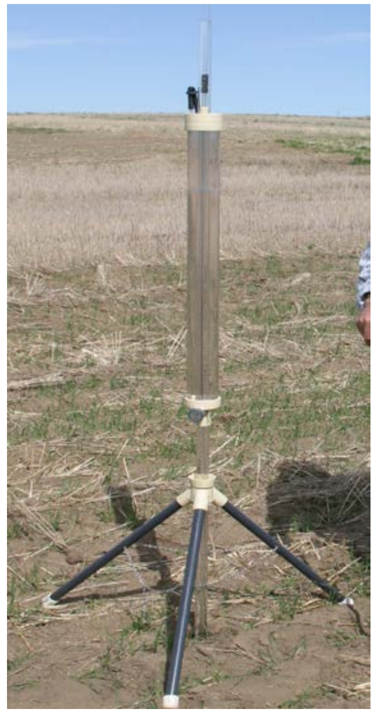

Figure C-3: Guelph Permeameter for Shallow Borehole Permeability

Photo Credit: USDA, 2005

C.6 Falling-Head Borehole Infiltration Test

The falling-head borehole infiltration test is commonly applied to assess infiltration at greater depths (e.g. 5 - 25 ft). The method is generally performed according to United States Bureau of Reclamation procedure 7300-89 (USBR, 1990). Caltrans has used the method to site stormwater infiltration structures (Caltrans, 2003). Essentially the method consists of boreholes, installing well casing with slots cut to release water at the target depths, backfilling the borehole, adding pre-soak water, and then filling again with water and recording the stage loss. An example diagram is shown in Figure C-4.

The testing procedures are summarized as follows:

1) Remove any smeared soil surfaces to provide a natural soil interface for testing the percolation of water. Remove all loose material. The U.S. EPA recommends scratching the sides with a sharp pointed instrument. (Note: upon tester's discretion, a 2-inch layer of coarse sand or fine gravel may be placed to protect the bottom from scouring and sediment.) Fill casing with clean water and allow to pre-soak for 24 hours or until the water has completely infiltrated.

2) Refill casing and monitor water level (distance from top of casing to top of water) for 1 hour. Repeat this procedure a total of four times. (Note: upon tester's discretion, the final field rate may either be the average of the four observations

or the value of the last observation. The final rate shall be reported in inches per hour.)

3) Testing may be done through a boring or open excavation.

4) The location of the test must be near the proposed facility.

5) Upon completion of the testing, the casings shall be immediately pulled and the test pit shall be back-filled.

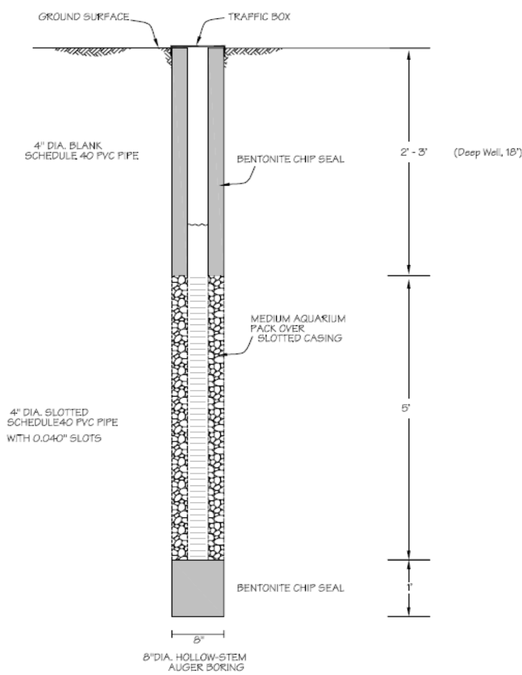

Figure C-4: Falling-Head Permeameter for Deep Borehole Permeability

Diagram Credit: Group Delta Consultants, 2008

C.7 Laboratory Soil Tests

If fill materials imported from off-site are part of an infiltration BMP design, a laboratory test is required to determine the infiltration rate of the fill soil. A sample of the fill soil from each area where a BMP will be located must be tested. The soil sample must be compacted to the same degree that will be present after final grading. Once prepared, the sample should be sent to a specialty laboratory to conduct a test of the infiltration rate. These results may then be used to assess the applicability of a specific BMP.

C.8 Assessment of Test Results

The results from field infiltration methods should be examined to consider data variability and sample distribution to determine if there has been adequate sampling. If the spatial variability (heterogeneity) is large, then additional field measurements may be necessary. The infiltration results should be compared to the information gathered on site soils and geology to see if they are consistent. The results of the site soils and infiltration testing may then be used in the siting, selection, sizing, and design of LID site design techniques and structural treatment BMPs.

C.9 References

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APPENDIX D : BMP PERFORMANCE GUIDANCE

D.1 Permit Requirement

Part 3, Section A.3 of Order R4-2010-0108 states the following:

> 3. *Each Permittee shall require that treatment control BMPs being implemented under the provisions of this Order shall be designed, at a minimum, to achieve the BMP performance criteria for storm water pollutants likely to be discharged as identified in Attachment "C", for an 85th percentile 24-hour runoff event determined as the maximized capture storm water volume for the area using a 48 to 72-hour draw down time, from the formula recommended in Urban Runoff Quality Management, WEF Manual of Practice No. 23/ASCE Manual of Practice No. 87, (1998). Expected BMP pollutant removal performance for effluent quality was developed from the WERF-ASCE/ U.S. EPA International BMP Database. Permittees shall select Treatment BMPs based on the primary class of pollutants likely to be discharged from the site/facility (e.g. metals from an auto repair shop). Permittees may develop guidance for appropriate Treatment BMPs for project type based on Attachment "C". For the treatment of pollutants causing impairments within the drainage of the impaired waterbody, permittees shall select BMPs from the top three performing BMP categories or alternative BMPs that are designed to meet or exceed the performance of the highest performing BMP for the pollutant causing impairment.*

Attachment C contains the following table:

Effluent Concentrations as Median Values

BMP Category	Total Suspended Solids (mg/L)	Total Nitrate-Nitrogen (mg/L)	Total Copper (µg/L)	Total Lead (µg/L)	Total Zinc (µg/L)
Detention Pond	27	0.48	15.9	14.6	58.7
Wet Pond	10	0.2	5.8	3.4	21.6
Wetland Basin	13	0.13	3.3	2.5	29.2
Biofilter	18	0.36	9.6	5.4	27.9
Media Filter	11	0.66	7.6	2.6	32.2
Hydrodynamic Device	23	0.29	11.8	5	75.1

Expected BMP pollutant performance for effluent quality was developed from the WERF-ASCE/U.S. EPA International BMP Database, 2007

D.2 Using Performance Statistics for BMP Selection

The observed performance of stormwater BMPs provides valuable quantitative information that can be used to infer the potential water quality benefits of stormwater BMP implementation. However, water quality data sets and the statistical methods used to summarize them inherently contain a high level of uncertainty. Consideration of this uncertainty is fundamental to the proper and responsible use of statistics. Some of the key issues that should be considered when drawing conclusions from data contained in the ASCE International BMP Database for the purposes of developing BMP selection guidance are discussed below.

Number of Representative BMPs

Some BMP types are not well represented in the ASCE International BMP Database due to small data sets. For example, the "Wetland Basin" category only included nine studies nationwide as compared to over 50 for biofilters at the time the data analysis was conducted for the MS4 permit (2007). For some pollutants, such as total copper, data are only available for four Wetland Basin studies. While the BMP Database continues to grow, there are currently less than 300 BMP studies included, with only approximately 50 in California. The size of the data set provides an indicator of the reliability of that data in representing the "typical" effluent concentration for that BMP type.

BMP Categorization

The BMP studies within the BMP database represent a wide spectrum of BMP types with a variety of designs and sizing criteria. While some guidance is provided on how to categorize BMPs, data providers are responsible for categorizing their own BMPs. Some of these BMPs could be poorly categorized due to a variety of reasons, such as differences in terminology, missing or inadequately sized treatment components (e.g., forebays, vegetation, or permanent pools) or variable treatment function (e.g., a seasonal wet pond). Ideally, the BMPs should be grouped according to common design components and/or sizing criteria, but there currently aren't enough data with design information to support such analyses. However, the BMP Database is currently undergoing a restructuring that is redefining or sub-categorizing the current BMP categories within the database.

Statistical Significant Difference between BMP Influent/Effluent

Some of the median effluent values reported in the BMP Database are not statistically different than the median influent values (i.e., no concentration reductions on average). No significant difference may indicate either low influent concentrations or poor performing BMPs for that pollutant. In either case, the effluent value alone would not be a reliable indicator of BMP performance. For example, as summarized in Geosyntec and Wright Water (2008), the data for Wetland Basins, a "top performing" BMP according to Attachment C of the MS4 permit, did not conclusively show statistically significant removals of TSS, nitrate-nitrogen, or total lead. Data for hydrodynamic separators and media filters indicate they are also ineffective at reducing nitrate-nitrogen concentrations.

Statistical Significant Differences in Effluent between BMP Types

The median effluent concentrations of the various BMP types are not necessarily statistically significantly different from each other. Statistical significance can be determined by analyzing whether the 95th percent confidence intervals overlap. The number of data points and the variability of those data points determine the confidence interval of each median value. If the effluent medians are not statistically significantly different from each other, it may not be possible to determine the "top three" performing BMPs as specified in the MS4 Permit. Confidence intervals about the median effluent concentrations for each BMP type are provided in Geosyntec and Wright Water (2008) (see attached).

D.3 Comparison of the Performance of Biofiltration BMPs and Retention BMPs

Background

Projects that demonstrate technical infeasibility for reducing EIA to ≤5% using Retention BMPs are eligible to use Biofiltration BMPs to achieve the EIA performance standard. Section 4.E.III.1.(b) of Order R4-2010-0108 states:

> *If on-site retention is determined to be technically infeasible pursuant to 4.E.III.2(b), an on-site biofiltration system that achieves equivalent stormwater volume and pollutant load reduction as would have been achieved by on-site retention shall satisfy the EIA limitation.*

Volume-based biofiltration BMPs shall be sized to treat 1.5 times the volume not retained using Retention BMPs. The remaining EIA requirement may also be satisfied with flow-based Biofiltration BMPs. Flow-based Biofiltration BMPs shall be sized for the remaining drainage area from which runoff must be retained (A_{Retain}) with a rainfall intensity that varies with time of concentration for the catchment tributary to the flow-based Biofiltration BMP, according to the following. Using this flow-based sizing method will achieve or exceed capture and treatment of 80% of the average annual runoff volume.

Time of Concentration, minutes	Design Intensity for 150% Sizing, in/hr
30	0.24
20	0.25
15	0.28
10	0.31
5	0.35

Methodology

A planning-level analysis was conducted to assess whether the range of Biofiltration BMPs included in the 2010 TGM, sized per these volume- or flow-based sizing criteria, would achieve equivalent pollutant load reduction to Retention BMPs. The following describes the step-wise method taken for the analysis.

Step 1: Estimate the Catchment Annual Load

Assumptions:

- Average Annual Rainfall- 14.5 inches (Oxnard Gauge) (precipitation, P)
- One acre Catchment (area, A)

Calculations:

1) Determine developed runoff coefficients for single-family, multi-family, commercial, and industrial land use types

    - Use average imperviousness values from Ventura Hydrology Manual (Exhibit 14B)
    - Assume soil group 2/3 (Group C soils) for pervious runoff coefficient (Cp, conservative value = 0.1)
    - Use developed runoff coefficient (C_d) equation from hydrology manual:

$$C_d = 0.95*(\text{imperviousness}) + (Cp)*(1\text{-imperviousness})$$

2) Calculate Average Annual Runoff Volume (cu-ft) using:

$$V_{avg\ annual} = C_d*(P/12)*A*43560$$

3) Multiply average annual runoff volume by respective event mean concentrations (EMCs) for pollutants of concern to get average annual loads.

    - Look at “EMC Arithmetic Means” to see EMCs by land use type.
    - EMCs calculated based on LA County Land Use specific data (LACDPW, 2000). Descriptive statistics estimated using the parametric bootstrap method suggested by Singh, Singh, and Engelhardt (1997).
    - Pollutants of concern: Total Suspended Solids (TSS), Total Copper, Total Zinc, and Total Nitrogen. TSS is representative of the sediment pollutant class as well as pollutants that are associated with particulates (e.g., total phosphorous, some metals, pesticides, some organics). Copper and zinc represent metals – lead has been removed from the environment using True Source Control (removal of lead from gasoline) and thus is not an important POC for Biofiltration BMP selection and design. Total nitrogen is representative in that it includes all of the species of nitrogen (organic nitrogen, ammonia, nitrate, and nitrite) and instead of focusing on one species (nitrate).

Step 2: Estimate Retention BMP Load Reduction

1) Determine Retention BMP Design volume:

- Design storm = 0.75”
- Use land use-based coefficients
- $V_{design} = C_d*(0.75/12)*A*43560$

2) Determine Retention BMP capture volume using CASQA 48-hour Drawdown Figure for Oxnard Gauge (CASQA, 2003)

   - Calculate Unit Basin Storage Volume using:
     - Unit Basin Storage Vol = V_{design}/ A
   - Using developed runoff coefficients, interpolate between runoff coefficient lines to determine the percentage of total runoff captured by Retention BMP.

3) Determine Annual Load Reduction

   - The percentage of the annual load that is reduced is the same as the percentage of runoff captured by the Retention BMP, assuming that all captured runoff is retained. The percent capture calculated in (2) can be multiplied by the catchment annual pollutant load to obtain the load reduction.

Step 3: Estimate Biofiltration BMP Load Reduction

1) Determine BMP Design volume as described in 2.a above, except:

   - Design storm = 1.5*0.75 = 1.125 inches

2) Determine BMP capture volume using CASQA 24-hour Drawdown Figure for Oxnard Gauge (CASQA, 2003) as described in 2.b. above

3) Determine annual load reduction. Load reduction in Biofiltration BMPs can occur via two pathways: incidental infiltration and treatment.

   - Incidental infiltration in Biofiltration BMPs was discussed in a publication by Strecker, Quigley, Urbonas, and Jones (Strecker et al, 2004). That study observed as much as 40% volume reduction through incidental infiltration. A recent summary of the studies in the ASCE BMP Database found the following average volume reductions: filter strips, 38%; vegetated swales, 48%; and bioretention with underdrain, 61% (Geosyntec, 2011; attached to this appendix).
   - Pollutant Load reduction via incidental infiltration can be calculated as follows (20% is the percent of the captured volume assumed to be reduced via incidental infiltration for this discussion):

Load reduced = Average annual Load * Percent Runoff Captured by BMP * 20%

- Load reduction through treatment calculated based on published literature on pollutant removals from biofiltration facilities.

- Load reduction through treatment is calculated as follows:

Load reduced = Average annual Load * Percent Runoff Captured by BMP *80% * Assumed Average Percent Removal

Note: 80% = 100%-20%, i.e. the captured runoff that was not infiltrated via incidental infiltration

Constituent	Range of Reported Removal Efficiencies from Literature[1]	Selected Removal Efficiency for Effectiveness Evaluation[2]	Selected Removal Efficiency for Enhanced Nitrogen Removal[3]
TSS	54-89	79	79
Total Zinc	48-96	77	77
Total Copper	33-92	72	72
Total Nitrogen	21-54	25	50

[1] Range of values from literature cited below:

1. Hererra Consultants and Geosyntec Consultants, 2010. Filterra® Bioretention Systems: Technical Basis for High Flow Rate Treatment and Evaluation of Stormwater Quality Performance. September 2010.
2. University of New Hampshire, 2009. University of New Hampshire Stormwater Center 2009 Biannual Report. www.unh.edu/erg/cstev.
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4. Brown, R.A., Hunt, W.F., and Kennedy, S.G., 2009. Designing Bioretention with an Internal Water Storage (IWS) Layer. Online at: http://www.bae.ncsu.edu/stormwater/PublicationFiles/IWS.BRC.2009.pdf.
5. Facility for Advancing Water Biofiltration. Online at: http://www.monash.edu.au/fawb/products/obtain.html.
6. Geosyntec Consultants and Wright Water Engineers, Inc., 2008. Overview of Performance by BMP Category and Common Pollutant Type, International Stormwater BMP Database Update. June 2008
7. Geosyntec Consultants and Wright Water Engineers, Inc., 2010. Categorical Summary of BMP Performance for Nutrient Concentration Data Contained in the International Stormwater BMP Database. December, 2010

[2] Removal efficiency for TSS, Total Zinc, and Total Copper represent average of values from literature. Removal efficiency for TN is that expected from a 'standard biofilter', that is, one not designed for enhanced nitrogen removal

[3] Removal efficiency for TN represented as average value of removals from bioretention systems with an anaerobic zone for enhanced removal of nitrogen

- The total load reduction is calculated as the sum of the reductions from these two pathways. The percent load reduction is calculated by dividing the total load reduction by the annual pollutant load from the catchment

Step 4: Comparison of Annual Load Reductions

1) Load reductions are compared by subtracting the load reduction calculated for Biofiltration BMPs from the load reduction calculated for Retention BMPs to determine the 'deficit' load reduction.

Results

Step 1: Estimate the Catchment Annual Load

1) Determine developed runoff coefficients for single-family, multi-family, commercial, and industrial land use types

Land Use	Imperviousness	Runoff Coefficient (C)
Single Family Residential	0.3	0.36
Multi Family Residential	0.69	0.69
Commercial	0.85	0.82
Industrial	0.93	0.89

2) Calculate Average Annual Runoff Volume (cu-ft), and

3) Multiply average annual runoff volume by respective event mean concentrations (EMCs) for pollutants of concern to get average annual loads.

Land Use	Arithmetic Means from Lognormal EMC Statistics			
	TSS (mg/L)	Total Zinc (mg/L)	Total Copper (mg/L)	Total Nitrogen (mg/L as N)
Single Family Residential	124.2	71.9	18.7	3.74
Multi Family Residential	39.9	125.1	12.1	3.31
Commercial	67	237.1	31.4	3.99
Industrial	219.2	537.4	34.5	3.74

Land Use	Average Annual Runoff Volume (cu-ft)	Catchment Pollutant Loads (kg/yr)			
		TSS	Total Zinc	Total Copper	Total Nitrogen
Single Family Residential	18,685	65,716	38	10	1,979
Multi Family Residential	36,134	40,826	128	12	3,387
Commercial	43,292	82,135	291	38	4,891
Industrial	46,871	290,933	713	46	4,964

Step 2: Estimate Retention BMP Load Reduction

1) Determine Retention BMP Design volume

Land Use	Design Volume (cu-ft)
Single Family Residential	967
Multi Family Residential	1869
Commercial	2239
Industrial	2424

2) Determine Retention BMP capture volume using CASQA 48-hour Drawdown Figure for Oxnard Gauge (CASQA, 2003)

Land Use	Design Volume (cu-ft)	Unit Basin Storage Volume (inches)	Approx % Capture
Single Family Residential	966	0.27	60.0%
Multi Family Residential	1,869	0.51	62.5%
Commercial	2,239	0.62	62.5%
Industrial	2,424	0.67	60.0%

3) Determine Annual Load Reduction

Land Use	Average Annual Pollutant Load Reduction (kg/yr) = Influent * Approx % Cap			
	TSS	Total Zinc	Total Copper	Total Nitrogen
Single Family Residential	39,429	23	5.9	1,187
Multi Family Residential	25,516	80	7.7	2,117
Commercial	51,335	182	24.1	3,057
Industrial	174,560	428	27.5	2,978

Land Use	Percent of Total Annual Loads			
	TSS	Total Zinc	Total Copper	Total Nitrogen
Single Family Residential	60.0%	60.0%	60.0%	60.0%
Multi Family Residential	62.5%	62.5%	62.5%	62.5%
Commercial	62.5%	62.5%	62.5%	62.5%
Industrial	60.0%	60.0%	60.0%	60.0%

Step 3: Estimate Biofiltration BMP Load Reduction

1) Determine Biofiltration BMP Design volume

Land Use	Design Volume (cu-ft)
Single Family Residential	1,450
Multi Family Residential	2,803
Commercial	3,359
Industrial	3,637

2) Determine BMP capture volume using CASQA 24-hour Drawdown Figure for Oxnard Gauge (CASQA, 2003)

Land Use	Design Volume (cu-ft)	Unit Basin Storage Volume (inches)	Approx % Capture
Single Family Residential	1,450	0.40	87.50%
Multi Family Residential	2,803	0.77	87.50%
Commercial	3,359	0.93	90.00%
Industrial	3,637	1.00	87.50%

3) Determine annual load reduction. Load reduction in Biofiltration BMPs can occur via two pathways: incidental infiltration and treatment.

Incidental Infiltration Scenario #1: 20% Volume Reduction

Land Use	Pollutant Load Reduction from 20% Incidental Infiltration (kg/yr)			
	TSS	Total Zinc	Total Copper	Total Nitrogen
Single Family Residential	11,500	7	2	346
Multi Family Residential	7,144	22	2	593
Commercial	14,784	52	7	880
Industrial	50,913	125	8	869

Land Use	Pollutant Load Reduction from Standard Treatment (kg/yr)				Enhanced Nitrogen Load Reduction (kg/yr)[1]
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	36,341	21	5	346	693
Multi Family Residential	22,577	69	6	593	1,185
Commercial	46,719	161	20	880	1,761
Industrial	160,886	384	23	869	1,737

[1] Anticipated removal if an anaerobic zone is provided for Enhanced Nitrogen removal.

Land Use	Total Pollutant Load Reduction from Standard Treatment + Incidental Infiltration (20%) (kg/yr)				Enhanced Nitrogen Load Reduction + Incidental Infiltration (20%) (kg/yr)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	47,841	27	6.7	693	1,039
Multi Family Residential	29,721	91	8.4	1,185	1,778
Commercial	61,503	213	26.8	1,761	2,641
Industrial	211,799	509	31.0	1,737	2,606

Land Use	Percent of Total Annual Loads from Standard Treatment + Incidental Infiltration (20%)				Enhanced Nitrogen % Load Reduction + Incidental Infiltration (20%)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	72.8%	71.4%	67.7%	35.0%	52.5%
Multi Family Residential	72.8%	71.4%	67.7%	35.0%	52.5%
Commercial	74.9%	73.4%	69.6%	36.0%	54.0%
Industrial	72.8%	71.4%	67.7%	35.0%	52.5%

Step 4: Comparison of Annual Load Reductions

Load reductions are compared by subtracting the load reduction calculated for Biofiltration BMPs from the load reduction calculated for Retention BMPs to determine the 'deficit' load reduction.

Land Use	Biofiltration Pollutant Load Reduction Deficit - Standard Treatment + Incidental Infiltration (20%) (kg/yr)				Enhanced Nitrogen + Incidental Infiltration (20%) Pollutant Load Reduction Deficit (kg/yr)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	-8,412	-4	-0.8	495	148
Multi Family Residential	-4,205	-11	-0.6	931	339
Commercial	-10,168	-32	-2.7	1,296	416
Industrial	-37,239	-81	-3.5	1,241	372

Note: a negative deficit means Biofiltration has a higher pollutant load reduction than Retention.

Land Use	Biofiltration Pollutant Load Reduction Deficit - Standard Treatment + Incidental Infiltration (20%) (%)				Enhanced Nitrogen + Incidental Infiltration (20%) Pollutant Load Reduction Deficit (%)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	-12.8%	-11.4%	-7.7%	25.0%	7.5%
Multi Family Residential	-10.3%	-8.9%	-5.2%	27.5%	10.0%
Commercial	-12.4%	-10.9%	-7.1%	26.5%	8.5%
Industrial	-12.8%	-11.4%	-7.7%	25.0%	7.5%

Conclusion: Biofiltration BMPs sized for 1.5 times the SQDV, with an average incidental infiltration of 20% of the average annual runoff volume, which is a conservative estimate of incidental infiltration for all types of Biofiltration Treatment Measures, provide equivalent pollutant load reduction to Retention BMPs for TSS and metals.

Incidental Infiltration Scenario #2: 40% Volume Reduction

Land Use	Pollutant Load Reduction from 40% Incidental Infiltration (kg/yr)			
	TSS	Total Zinc	Total Copper	Total Nitrogen
Single Family Residential	23,000	13	3	693
Multi Family Residential	14,289	45	4	1,185
Commercial	29,569	105	14	1,761
Industrial	101,827	250	16	1,737

Land Use	Pollutant Load Reduction from Standard Treatment (kg/yr)				Enhanced Nitrogen Load Reduction (kg/yr)[1]
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	27,256	15	3.7	260	519
Multi Family Residential	16,932	52	4.7	445	889
Commercial	35,039	121	14.9	660	1,321
Industrial	120,665	288	17.2	652	1,303

[1] Anticipated removal if an anaerobic zone is provided for Enhanced Nitrogen removal.

Land Use	Total Pollutant Load Reduction from Standard Treatment + Incidental Infiltration (40%) (kg/yr)				Enhanced Nitrogen Load Reduction + Incidental Infiltration (40%) (kg/yr)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	50,256	29	7.2	952	1,212
Multi Family Residential	31,221	97	9.0	1,630	2,074
Commercial	64,608	225	28.8	2,421	3,082
Industrial	222,491	538	33.3	2,389	3,040

Land Use	Percent of Total Annual Loads from Standard Treatment + Incidental Infiltration (40%)				Enhanced Nitrogen % Load Reduction + Incidental Infiltration (40%)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	76.5%	75.4%	72.6%	48.1%	61.3%
Multi Family Residential	76.5%	75.4%	72.6%	48.1%	61.3%
Commercial	78.7%	77.6%	74.7%	49.5%	63.0%
Industrial	76.5%	75.4%	72.6%	48.1%	61.3%

Step 4: Comparison of Annual Load Reductions

Load reductions are compared by subtracting the load reduction calculated for Biofiltration BMPs from the load reduction calculated for Retention BMPs to determine the 'deficit' load reduction.

Land Use	Biofiltration Pollutant Load Reduction Deficit - Standard Treatment + Incidental Infiltration (40%) (kg/yr)				Enhanced Nitrogen + Incidental Infiltration (40%) Pollutant Load Reduction Deficit (kg/yr)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	-10,827	-6	-1.2	235	-25
Multi Family Residential	-5,705	-17	-1.3	487	42
Commercial	-13,273	-44	-4.7	636	-24
Industrial	-47,931	-110	-5.8	589	-62

Note: a negative deficit means Biofiltration has a higher pollutant load reduction than Retention.

Land Use	Biofiltration Pollutant Load Reduction Deficit - Standard Treatment + Incidental Infiltration (40%) (%)				Enhanced Nitrogen + Incidental Infiltration (40%) Pollutant Load Reduction Deficit (%)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	-16.5%	-15.4%	-12.6%	11.9%	-1.3%
Multi Family Residential	-14.0%	-12.9%	-10.1%	14.4%	1.2%
Commercial	-16.2%	-15.1%	-12.2%	13.0%	-0.5%
Industrial	-16.5%	-15.4%	-12.6%	11.9%	-1.3%

Conclusion: Biofiltration BMPs sized for 1.5 times the SQDV, with an average incidental infiltration of 40% of the average annual runoff volume, which is representative of vegetated swales and filter strips, provide equivalent pollutant load reduction to Retention BMPs for all of the pollutants of concern.

Incidental Infiltration Scenario #3: 60% Volume Reduction

Land Use	Pollutant Load Reduction from 60% Incidental Infiltration (kg/yr)			
	TSS	Total Zinc	Total Copper	Total Nitrogen
Single Family Residential	34,501	20	5	1,039
Multi Family Residential	21,433	67	6	1,778
Commercial	44,353	157	21	2,641
Industrial	152,740	374	24	2,606

Land Use	Pollutant Load Reduction from Standard Treatment (kg/yr)				Enhanced Nitrogen Load Reduction (kg/yr)[1]
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	18,170	10	2	173	346
Multi Family Residential	11,288	34	3	296	593
Commercial	23,359	81	10	440	880
Industrial	80,443	192	11	434	869

[1] Anticipated removal if an anaerobic zone is provided for Enhanced Nitrogen removal.

Land Use	Total Pollutant Load Reduction from Standard Treatment + Incidental Infiltration (60%) (kg/yr)				Enhanced Nitrogen Load Reduction + Incidental Infiltration (60%) (kg/yr)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	52,671	30	7.7	1,212	1,385
Multi Family Residential	32,722	102	9.6	2,074	2,371
Commercial	67,712	238	30.7	3,082	3,522
Industrial	233,183	567	35.5	3,040	3,475

Land Use	Percent of Total Annual Loads from Standard Treatment + Incidental Infiltration (60%)				Enhanced Nitrogen % Load Reduction + Incidental Infiltration (60%)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	80.2%	79.5%	77.6%	61.3%	70.0%
Multi Family Residential	80.2%	79.5%	77.6%	61.3%	70.0%
Commercial	82.4%	81.7%	79.8%	63.0%	72.0%
Industrial	80.2%	79.5%	77.6%	61.3%	70.0%

Step 4: Comparison of Annual Load Reductions

Load reductions are compared by subtracting the load reduction calculated for Biofiltration BMPs from the load reduction calculated for Retention BMPs to determine the 'deficit' load reduction.

Land Use	Biofiltration Pollutant Load Reduction Deficit - Standard Treatment + Incidental Infiltration (60%) (kg/yr)				Enhanced Nitrogen + Incidental Infiltration (60%) Pollutant Load Reduction Deficit (kg/yr)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	-13,242	-7	-1.7	-25	-198
Multi Family Residential	-7,206	-22	-1.9	42	-254
Commercial	-16,378	-56	-6.7	-24	-465
Industrial	-58,623	-139	-8.1	-62	-496

Note: a negative deficit means Biofiltration has a higher pollutant load reduction than Retention.

Land Use	Biofiltration Pollutant Load Reduction Deficit - Standard Treatment + Incidental Infiltration (60%) (%)				Enhanced Nitrogen + Incidental Infiltration (60%) Pollutant Load Reduction Deficit (%)
	TSS	Total Zinc	Total Copper	Total Nitrogen	Total Nitrogen
Single Family Residential	-20.2%	-19.5%	-17.6%	-1.3%	-10.0%
Multi Family Residential	-17.7%	-17.0%	-15.1%	1.2%	-7.5%
Commercial	-19.9%	-19.2%	-17.3%	-0.5%	-9.5%
Industrial	-20.2%	-19.5%	-17.6%	-1.3%	-10.0%

Conclusion: Biofiltration BMPs sized for 1.5 times the SQDV, with an average incidental infiltration of 60% of the average annual runoff volume, which is representative of bioretention with an underdrain, is equivalent to or exceeds the pollutant load reduction of Retention BMPs for all of the pollutants of concern.

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APPENDIX E : BMP SIZING WORKSHEETS

E.1 Structural Treatment BMP Sizing Criteria

The BMP sizing criteria for determining the design volume or design flow for a proposed BMP are discussed in this appendix. These criteria must be used for all stormwater BMPs installed in new and re-development projects in Ventura County. This section outlines the rainfall analyses, Ventura County MS4 Permit sizing criteria, and recommended sizing methods for both volumetric and flow-based analysis.

Sizing Criteria

The type of rainfall analysis required depends on whether the BMP is a volume-based or flow-based BMP. This distinction between volume-based and flow-based controls is not always clear, especially in a sequence of BMPs or a treatment train. The following are general guidelines for each type of control.

- Volume-based BMPs are designed to treat a volume of runoff, which is detained for a certain period of time to allow for the settling of solids and associated pollutants. Volume-based BMPs included in this manual are bioretention, planter boxes, infiltration systems, and retention/detention BMPs.

- Flow-based BMPs treat water on a continuous flow basis. Flow-based BMPs included in this manual are vegetated swales, filter strips, filtration systems, and hydrodynamic devices.

The four volume-based and three flow-based BMP sizing criteria included in the Ventura County MS4 Permit (Order No. 09-0057) are included below.

The water quality design volume for volume-based BMPs must be determined using one of the following options:

1) The 85th percentile 24-hour runoff event determined as the maximized capture stormwater volume for the area using a 48 to 72-hour draw down time, from the formula recommended in Urban Runoff Quality Management, WEF Manual of Practice No. 23/ASCE Manual of Practice No. 87, (1998); or

2) The volume of annual runoff based on unit basin storage water quality volume to achieve 80 percent or more volume treatment; or

3) The volume of runoff produced from a 0.75 inch storm event; or

4) 80 percent of the average runoff volume using an appropriate public domain continuous flow model [such as Storm Water Management Model (SWMM) or Hydrologic Engineering Center – Hydrologic Simulation Program – Fortran (HEC-HSPF)], using the local rainfall record and relevant BMP sizing and design data.

Flow-based BMPs must be designed to capture and treat the water quality design flow rate generated from one of the following criterion:

1) The flow of runoff produced from a rain event equal to at least 0.2 inches per hour intensity; or

2) The flow of runoff produced from a rain event equal to at least 2 times the 85th percentile hourly rainfall intensity as determined from local rainfall records; or

3) Eight percent of the 50-year storm design flow rate as determined from the method provided below.

These sizing methods are explained below.

Methods for Determining the Water Quality Design Volume

Method 1: Urban Runoff Quality Management (URQM) Approach

The volume-based BMP sizing methodology described in Urban Runoff Quality Management (WEF Manual of Practice No. 23/ASCE Manual of Practice No. 87, (1998), pages 175-178) estimates the "maximized stormwater quality capture volume." The URQM approach is based on the translation of rainfall to runoff using two regression equations. The first regression equation, which relates rainfall to runoff, was developed using two years of data from more than 60 urban watersheds nationwide. The second regression equation relates mean annual runoff-producing rainfall depths to the "Maximized Water Quality Capture Volume" which corresponds to the "knee of the cumulative probability curve". This second regression was based on analysis of long-term rainfall data from seven rain gages representing climatic zones across the country. The Maximized Water Quality Capture Volume corresponds to approximately the 85th percentile runoff event, and ranges from 82 to 88%.

The two regression equations that form the URQM approach are as follows:

$$C = 0.858imp^3 - 0.78imp^2 + 0.774imp + 0.04 \quad \text{(Equation E-1)}$$

$$P_o = (a \cdot C) \cdot P_6 \quad \text{(Equation E-2)}$$

Where:

C	=	watershed runoff coefficient (unitless)
imp	=	watershed impervious ratio which is equal to the percent total imperviousness divided by 100 (ranges from 0 to 1)
P_o	=	maximized detention storage volume based on the volume capture ratio as its basis (watershed inches)

a	=	regression constant from least-squares analysis (unit less), a=1.582 and a=1.963 for 24 and 48 hour draw down, respectively
P_6	=	mean storm precipitation volume (watershed inches)

P_6 can be determined by two ways: Figure 5.3 in Urban Runoff Quality Management, or by performing analysis on local historical rainfall data. To determine the mean precipitation, EPA's Synoptic Rainfall Analysis Program – SYNOP – can be applied (see *Other Rainfall Analysis Methods* below).

The runoff coefficient equation in the URQM approach (Method 1) is not appropriate for the California BMP Handbook approach (Method 2), as Equation E-4 was developed in conjunction with the regression constants used in Method 1.

Method 2: Treatment of 80% or more of the Total Volume

Most water quality facilities are designed to treat only a portion of the runoff from a given site, as it is not economically feasible to capture 100% of the runoff. The percent of runoff treated by a basin is referred to as the "percent capture". There are a number of methods which allow calculation of the percent capture, including the California Stormwater Quality Association (CASQA) method (recommended by the 2002 Ventura County Manual), and using the EPA Stormwater Management Model (SWMM).

CASQA Method

The California Stormwater Quality Association (CASQA) BMP Handbook method estimates the basin volume to achieve various levels of volume capture (e.g., 80% for this sizing criterion). In the CASQA BMP Handbook New Development and Redevelopment (2003), a proprietary version of the Storage, Treatment, Overflow, Runoff Model (STORM) is used as the basis for the volume-based BMP sizing criteria. The model results are presented as the relationship between "unit basin storage volume" and "% volume capture" of the BMP", varying with drawdown time and runoff coefficient. Knowing the drawdown time, the runoff coefficient, and the desired percent capture will yield the "unit basin storage volume". The "unit basin storage volume" can then be used to size the BMP using the following equation (note that "unit basin storage volume" is given in inches, so units will have to be adjusted accordingly):

$$\text{BMP Volume} = \text{Unit Basin Storage Volume} \times \text{Tributary Area} \qquad \text{(Equation E-3)}$$

Results for several rain gauges are presented in Appendix D of the CASQA BMP Handbook New Development and Redevelopment (CASQA, 2003). Results are provided for a range of runoff coefficients and for 24 hour and 48 hour drawn down times. In order to use the curves provided in Appendix D, it is necessary to know the runoff coefficient for the area tributary to the BMP, the drawn down time (a.k.a. drain time) of the facility, and the percent capture goal (e.g., 80%).

Drawdown time is the time required to drain a facility that has reached its design capacity; usually expressed in hours. Drain time is important as it is a surrogate for residence time, which affects the particle settling in the basin. Estimates for design drain time vary, and ideally would be determined based on site-specific information on the size, shape, and density or settling velocity of suspended particulates in the runoff. Because this information is generally not available for a specific site, estimates of appropriate ranges for settling time have generally relied on settling column test information reported in the literature.

An important source of drain time information is settling column tests conducted by Grizzard et. al. (1986) as part of the Nationwide Urban Runoff Program (NURP). Grizzard found that settling times of 48 hours resulted in removals of 80% to 90% of total suspended solids (TSS). Rapid initial removal was also observed in stormwater samples with medium (100 to 215 mg/L) and high (721 mg/L) initial TSS concentrations. For example, at settling times of 24 hours, the 80% to 90% removals were already achieved in samples with medium and high initial TSS, whereas only 50% to 60% removal was achieved in those with low initial TSS.

Given the data provided above, a drain time of 36 to 48 hours is recommended for sizing volume-based BMPs. This is also consistent with the recommendation of vector control agencies that structures be designed to drain in less than 72 hours to minimize mosquito breeding.

The rain gauge that is recommended for use for the area permitted by the Ventura county MS4 Permit (Order No. 09-0057) is the Oxnard Equipment Yard Gauge (168), which has a 40 year rainfall record. The graph included in the CASQA handbook can be seen in Figure E-1 below.

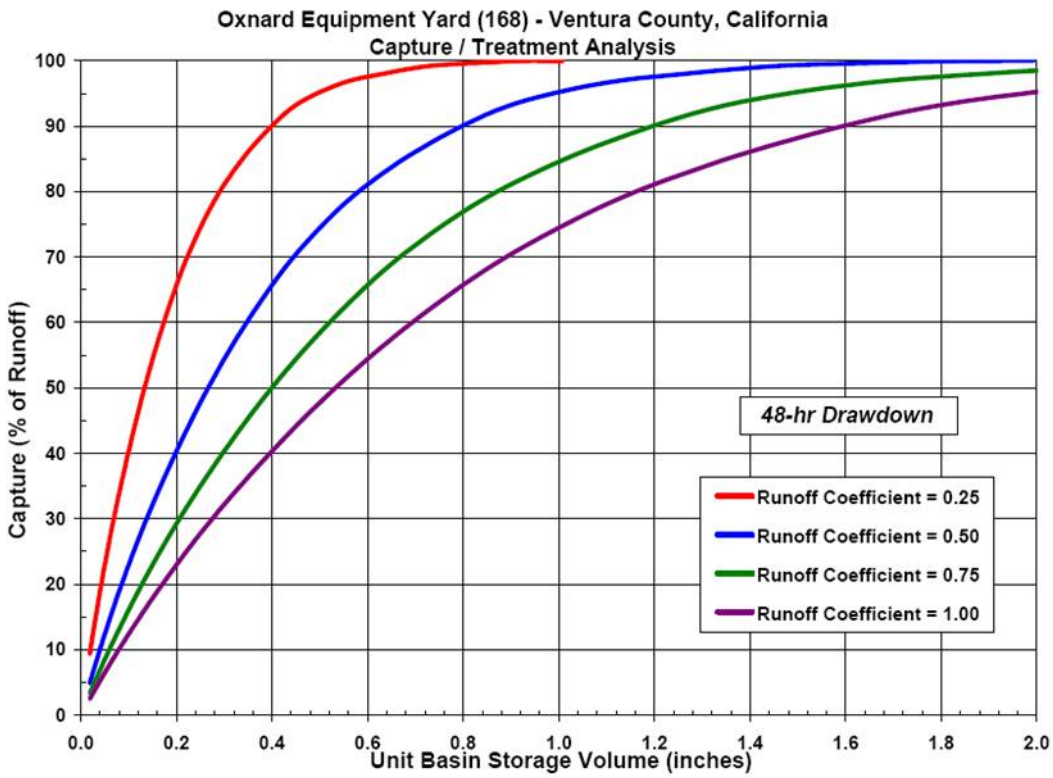


Figure E-1: CASQA 48-hour Drawdown Figure for Oxnard Gauge

This method has been modified for Ventura County. To use this method, follow the calculation procedure below. This refers to Figure E-3.

Ventura County Calculation Procedure

1) Review the area draining to the proposed treatment control measure. Determine the effective imperviousness (I_{WQ}) of the drainage area.

2) Estimate the total imperviousness (impervious percentage) of the site by the determining the weighted average of individual areas of like imperviousness.

3) Enter Figure E-2 along the horizontal axis with the value of total imperviousness calculated in Step 1. Move vertically up Figure E-2 until the appropriate curve (G-5.1 (filter strip) or G-5.2 (vegetated swale) employed individually or G-5.1 and G-5.2 employed together) is intercepted. Move horizontally across Figure E-2 until the vertical axis is intercepted. Read the Effective Imperviousness value along the vertical axis.

4) Note that if G-5.1 and/or G-5.2 are implemented on only a portion of the site, the site may be divided and effective imperviousness determined for the portion of the site for which site design controls have been implemented. The resulting effective imperviousness may be combined with total imperviousness of the

remainder of the site to determine a weighted average total imperviousness for the entire site.

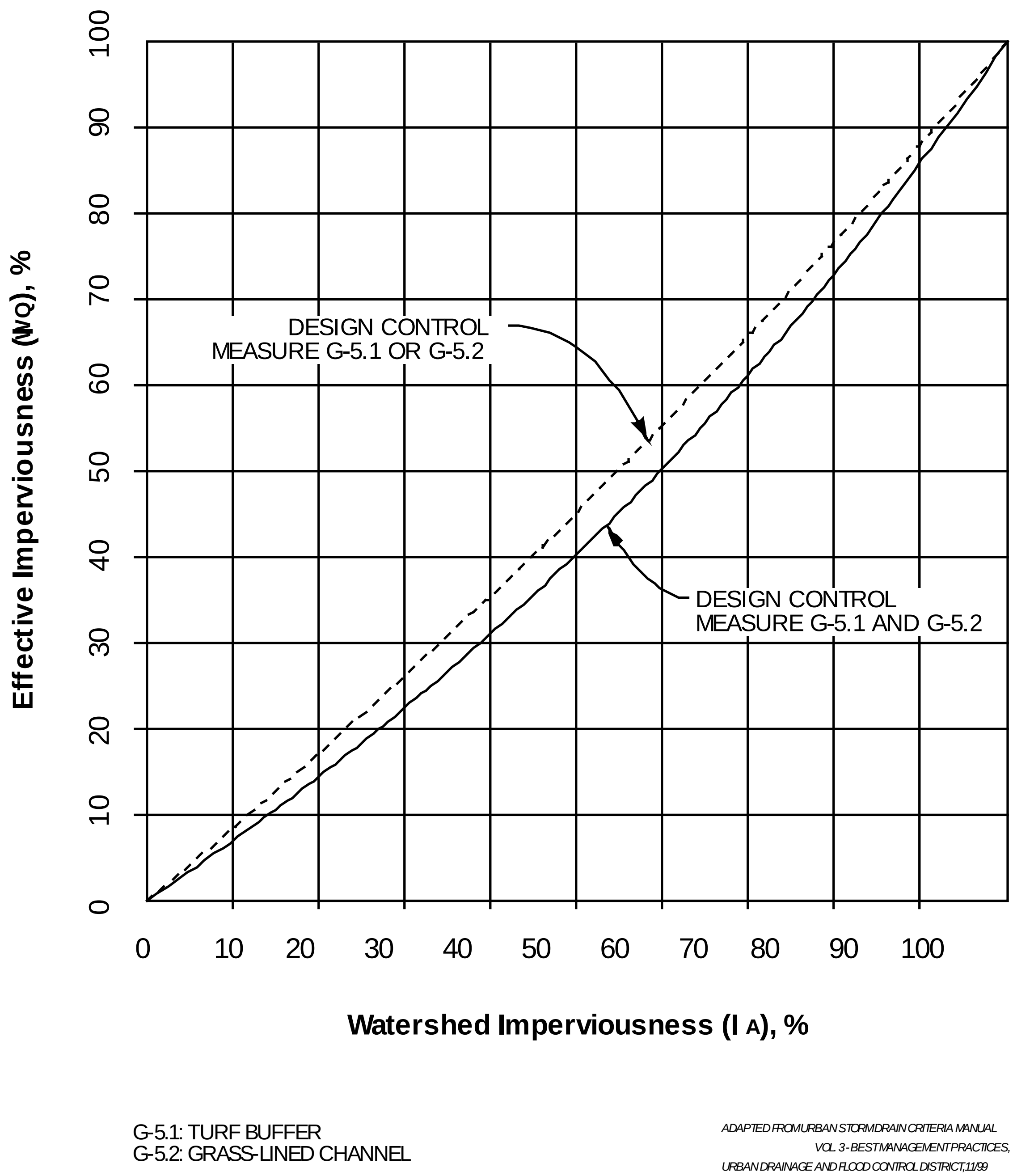


Figure E-2: Effective Imperviousness based on Watershed Imperviousness

5) Figure E-3 provides a direct reading of Unit Basin Storage Volumes required for 80% annual capture of runoff for values of "I_{WQ}" determined in Step 1. Enter the horizontal axis of Figure E-3 with the "I_{WQ}" value from Step 1. Move vertically up

Figure E-3 until the appropriate drawdown period line is intercepted. (The design drawdown period specified in the respective Fact Sheet for the proposed treatment control measure.) Move horizontally across Figure E-3 from this point until the vertical axis is intercepted. Read the Unit Basin Storage Volume along the vertical axis.

6) Figure E-3 is based on Precipitation Gage 168, Oxnard Airport. This gage has a data record of approximately 40 years of hourly readings and is maintained by Ventura County Flood Control District. Figure E-3 is for use only in the permit area specified in Regional Board Order No. 00-108, NPDES Permit No. CAS004002.

7) The SQDV for the proposed treatment control measure is then calculated by multiplying the Unit Basin Storage Volume by the contributing drainage area. Due to the mixed units that result (e.g., acre-inches, acre-feet) it is recommended that the resulting volume be converted to cubic feet for use during design.

Example Stormwater Quality Design Volume Calculation

1) Determine the drainage area contributing to control measure, A_t. Example: 10 acres.

2) Determine the area of impervious surfaces in the drainage area, A_i. Example: 6.4 acres.

3) Calculate the percentage of impervious, $I_A = (A_i / A_t)*100$

   Example:

   Percent Imperviousness = $(A_i / A_t)*100$ = (6.4 acres/10 acres)*100 = 64%

4) Determine Effective Imperviousness using Figure 3-4.

   $I_{WQ} = 60\%$

5) Determine design drawdown period for proposed control measure.

6) Determine the Unit Basin Storage Volume for 80% Annual Capture, V_u using Figure E-3.

   For $I_{WQ}/100 = 0.60$ and drawdown = 40 hrs, $V_u = 0.64$ in.

7) Calculate the volume of the basin, V_b, where

   $V_b = V_u * A_t$. (Equation E-4)

Where

V_b = Volume of basin

V_u = Unit basin storage volume

A_t = Total tributary area

8) V_b = (0.64 in)(10 ac)(ft/12 in(43,560 ft^2 / ac) = 23,232 ft^3.

9) Solution: Size the proposed control measure for 23,232 ft^3 and 40-hour drawdown.

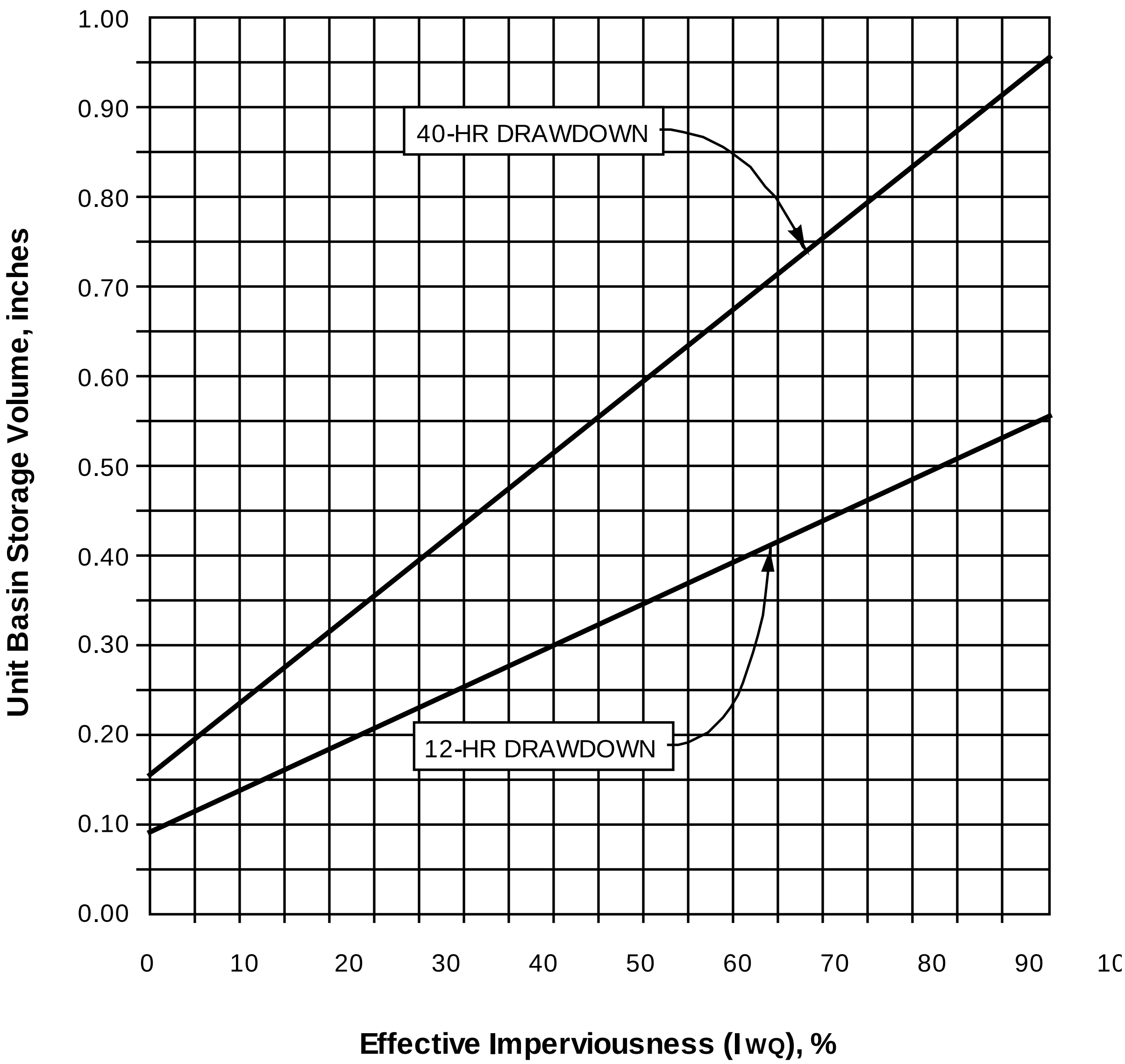


Figure E-3: Unit Basin Storage Volume for Design Volume Method 2

Method 3: 0.75 Inch Design Storm Approach

Equation E-8 can be used to determine the water quality design volume for Method 3.

Calculation Procedure

1) Determine the area from which runoff must be retained on-site (A_{retain}) using the method below:

   The allowable EIA for a project site can be calculated as follows:

   $EIA_{allowable} = (A_{project})*(\%_{allowable})$ (Equation E-5)

Where:

$EIA_{allowable}$	=	the maximum impervious area from which runoff can be treated and discharged off-site [and not retained on-site] (acres).
$A_{project}$	=	the total project area (acres). "Total project area" for new development and redevelopment projects is defined as the disturbed, developed, and undisturbed portions within the project's property (or properties) boundary, at the project scale submitted for first approval.
$\%_{allowable}$	=	ranges from 5 percent to 30 percent, based on a project specific assessment of technical feasibility for retaining runoff and whether the project is located in an existing urban area.

The drainage area from which Project generated runoff must be retained on-site is the total impervious area minus the $EIA_{allowable}$, which can be calculated as follows:

$A_{retain} = TIA - EIA_{allowable} = (P*A_{project}) - EIA_{allowable}$ (Equation E-6)

Where:

A_{retain}	=	the drainage area from which runoff must be retained (acres)
TIA	=	total impervious area (acres)
$EIA_{allowable}$	=	the maximum impervious area from which runoff can be treated and discharged off-site [and not retained on-site] (acres).
P	=	imperviousness of project area (%)/100
$A_{project}$	=	the total project area (acres)

Calculation Procedure

1) Determine the area from which runoff must be retained on-site (A_{retain}) using method above.

2) Determine the runoff coefficient per the following method:

$$C = 0.95*imp + C_p (1-imp) \qquad \text{(Equation E-7)}$$

Where:

C	=	runoff coefficient
imp	=	impervious fraction of watershed
C_p	=	pervious runoff coefficient, determined using table below

Table E-1: Pervious Runoff Coefficient Based on Ventura Soil Type

Ventura Soil Type (Soil Number)	Cp value
1	0.15
2	0.10
3	0.10
4	0.05
5	0.05
6	0
7	0

3) The volume can be calculated using equation E-8 below:

$$SQDV = C*(0.75/12)*A_{retain} \qquad \text{(Equation E-8)}$$

Where:

SQDV	=	the water quality design volume (acre-feet)
C_{imp}	=	runoff coefficient, calculated by equation (4) above
0.75	=	the design rainfall depth (in) [based on sizing method (c)]
A_{retain}	=	the drainage area from which runoff must be retained (acres)

Method 4: 80 percent of the average runoff volume using an appropriate public domain continuous flow model

Models that can be used for this calculation include the Storm Water Management Model (SWMM) or Hydrologic Engineering Center – Hydrologic Simulation Program – Fortran (HEC-HSPF)], using the local rainfall record and relevant BMP sizing and design data.

Sizing Method 4 allows for alternative sizing methods to be used as long as the selected method produces a water quality design volume based on historical rainfall records that achieves 80% capture of the average runoff volume. While sizing Methods 2 and 3 are appropriate for low lying areas within Ventura County, continuous simulation (using historical rainfall record) is well suited to sizing BMPs in locations with higher average rainfall. This method is the recommended sizing method for Ventura County, using appropriate local data inputs. For BMP locations at higher elevations, with larger rainfall, Method 1 is also better suited to sizing volume-based BMPs using rainfall representative of the site where the BMP will be located.

Continuous runoff modeling takes a long, uninterrupted record of observed rainfall data and transforms it into a record of runoff data. This is done by use of a set of mathematical algorithms that represent the rainfall-runoff processes. EPA's Stormwater Management Model (U.S. EPA, 2000) (SWMM) is one type of continuous runoff model. The runoff module of SWMM subdivides each drainage area into two inclined planes, one for impervious areas and one for pervious areas. Manning's equation is applied to estimate runoff taking into account rainfall intensity, initial losses, evapotranspiration, and infiltration (for pervious areas). The width and length of each plane is selected based on the drainage area configuration and existing and proposed drainage features. Hourly rainfall data is the primary model input for generating runoff volumes and rates. Additional input data are required to characterize imperviousness, soils, topography, and losses associated with evapotranspiration, infiltration, and initial losses.

Sizing BMPs using this type of alternative should only be conducted by qualified personnel with a thorough understanding of the simulated hydrologic processes and operation of the selected hydrology model.

Methods for Determining the Water Quality Design Flow

Each of the flow-based sizing alternatives is described in detail below.

Method 1: Runoff Produced by 0.2 Inches per Hour Rainfall Intensity

The rainfall analysis for flow-based controls focuses on estimating the design rainfall intensity, which is then converted to a design flow rate using the rational method shown in Equation E-9.

$$SQDF = CiA \qquad \text{(Equation E-9)}$$

Where:

SQDF	=	design flow rate (cfs)
C	=	runoff coefficient, calculated with the Ventura County Hydrology Manual method (see Equation E-5) (unitless)
i	=	rainfall intensity (in/hr) (0.2 in/hr)
A	=	watershed area (acres)

Note that 1 acre-in/hr = 1.0083 cfs; this conversion factor can be used with Equation D-9, but is not necessary as the uncertainty for the other parameters is generally well above 0.8%.

Method 2: Runoff Produced by Twice the 85th Percentile Rainfall Intensity

This method is analogous to the rational method used in Method 1, except that twice the historical 85th percentile rainfall intensity for the site location is used for the design rainfall intensity. This method is expected to result in a higher design rainfall intensity and design flow rate compared to Method 1 for most of the rain gages in the District.

Method 3: Runoff Produced by eight percent of the 50-year storm design flow rate

The Stormwater Quality Design Flow (SQDF) is defined to be equal to 8 percent of the peak rate of runoff flow from the 50-year storm as determined using the procedures set forth in the *Hydrology Manual*.

Calculation Procedure

1) The Stormwater Quality Design Flow (SQDF) in Ventura County is defined as SQDF

2) Calculate the peak rate of flow from the 50-year storm ($Q_{P,\ 50\ yr.}$) using the procedures set forth in the *Hydrology Manual* or as directed by the local agency Drainage Master Plan.

3) Convert $Q_{P,\ 50yr}$ (Step 2) to $Q_{P,\ SQDF}$ (Step 1).

$$Q_{P,\ SQDF} = 0.1 \times Q_{P,\ 50yr} \qquad \text{(Equation E-10)}$$

Example Stormwater Quality Design Flow Calculation

The steps below illustrate calculation of SQDF:

1) Calculate the peak rate of flow from a 50-year storm.

$Q_{p,\ 50\ yr.}$ = 10 cfs from the *Ventura County Hydrology Manual*

4) Convert $Q_{p,50\ yr}$ (Step 2) to $Q_{p,\ SQDF}$ (Step 1)

   SQDF = 0.8 x 10 cfs (Equation E-11)

   SQDF = 0.8 cfs

Rainfall Analysis Methods

The rainfall analysis methods listed below have the benefits of including the most recent rainfall data. Additionally, if the site is not close to an isohyet map rainfall gauge, these methods may be more accurate due to the variability of rainfall due to changing microclimates caused by elevation and distance from the ocean.

A resource available for obtaining rainfall data in Ventura County is the data collected and compiled by the National Climatic Data Center (NCDC).

There are many NCDC stations within Ventura County that collect or have collected hourly precipitation data. Some of these stations are no longer in operation and others may not have a sufficiently long period of record over which precipitation data has been collected to be of use for properly sizing treatment BMPs. NCDC data may be obtained online at the NCDC website http://www.ncdc.noaa.gov/oa/ncdc.html.

Rainfall Analysis Using EPA'S SYNOP Program

US EPA's Synoptic Rainfall Data Analysis Program (SYNOP) aggregates hourly rainfall data into individual storm events and computes event descriptive statistics. The SYNOP program calculates the duration, volume, and intensity for individual storms as well as average annual statistics. Recurrence interval and probability results are also available as output options. The SYNOP program allows the user to screen out storms that are not expected to result in runoff (see step 2 below).

The SYNOP rainfall analysis is conducted to output event-specific data in addition to average annual statistics. The individual storm event data can be ranked to give the 85th percentile storm or averaged to give the mean storm size.

Steps for conducting SYNOP rainfall analysis are as follows:

1) Obtain the hourly rainfall data for the gage of interest from the NCDC or other agency.

2) Run SYNOP for the available rain gage data. Model input parameters include the inter-event time and a minimum storm event size. The inter-event time specifies the minimum duration in which precipitation does not occur, used to define separate storm events, while the minimum storm event is the depth of precipitation generated by a storm below which runoff generally does not occur. Typically, an inter-event time of 6 hours (USEPA, 1989), and a minimum storm

event size of 0.10 inches are used (i.e., storms of 0.10 inches or less are not considered to produce runoff typically). Model results include event-specific and annual statistics during the period of record analyzed.

3) Rank and average the SYNOP storm event output.

References

California Stormwater Quality Association, 2003. Stormwater Best Management Practice Handbook, New Development and Redevelopment, January 2003. http://www.cabmphandbooks.com/

Grizzard T.J., C.W. Randall, B.L. Weand, and K.L. Ellis (1986). Effectiveness of Extended Detention Ponds, in Urban Runoff Quality – Impact and Quality Enhancement Technology: pp. 323-337.

Schueler, T., 1987. “Controlling Urban Runoff: A Practical Manual for Planning and Designing Urban BMPs,” Publication No. 87703, Metropolitan Washington Council of Governments, Washington, DC.

USEPA, Driscoll, E.D., E. Strecker, G. Palhegyi, 1989. Analysis of Storm Events, Characteristics for Selected Rain Gauges throughout the United States.

WEF Manual of Practice No. 23/ASCE Manual and Report on Engineering Practice No. 87, 1998: Urban Runoff Quality Management.

E.2 INF-1 Infiltration Basin/ INF-2 Infiltration Trench/ INF-4 Drywell

This worksheet can be used for sizing INF-1 Infiltration Basins, INF-2 Infiltration Trenches, or INF-4 drywells. An infiltration basin is an earthen basin constructed into naturally pervious soils which retains the SQDV and allows the retained runoff to percolate into the underlying native soils over a specified period of time. Infiltration trenches are long, narrow, gravel-filled trenches, often vegetated, that infiltrate stormwater runoff from small drainage areas. Drywells are similar to infiltration trenches, but the geometry and materials are slightly different. A dry well may be either a small excavated pit filled with aggregate or a prefabricated storage chamber or pipe segment, with the depth of the drywell greater than the width.

Sizing Methodology

Infiltration facilities can be sized using one of two methods: a simple sizing method or a routing modeling method. With either method the SQDV volume must be completely infiltrated within 12 to 72 hours (see Appendix E, Section E.1 for a discussion on drawdown time and BMP performance). The simple sizing procedures provided below can be used for either infiltration basins, infiltration trenches (see INF-2: Infiltration Trench) or drywells (INF-4: Drywell). For the routing modeling method, refer to VEG-8 Sand Filters.

Step 1: Calculate the design volume

Infiltration facilities shall be sized to capture and infiltrate the SQDV volume (see Section 2 and Appendix E) with a 12 - 72 hour drawdown time (see Appendix E, Section E.1).

Step 2: Determine the Design Percolation Rate

The percolation rate will decline between maintenance cycles as particulates accumulate in the infiltrative layer and the surface becomes occluded. Additionally, monitoring of actual facility performance has shown that the full-scale infiltration rate is far lower than the rate measured by small-scale testing. It is important that adequate conservatism is incorporated in the selection of design percolation rates. For infiltration trenches, the design percolation rate discussed here is the percolation rate of the underlying soils, which will ultimately drive infiltration through the trench, and not the percolation rate of the filter media bed (refer to the "Geometry and Sizing" section of INF-2 for the recommended composition of the filter media bed for infiltration trenches). See INF-1: Infiltration Basin for guidance in developing design percolation rate correction factors.

Step 3: Calculate Surface Area

Determine the size of the required infiltrating surface by assuming the SQDV will fill the available ponding depth plus (for infiltration trenches/ drywells with aggregate) the void spaces within the filter media based on the computed porosity of the media (normally about 32%).

1) Determine the maximum depth of runoff that can be infiltrated within the required drain time as follows:

$$d_{\max} = \frac{P_{design}}{12} t \qquad \text{(Equation E-12)}$$

Where:

d_{max} = the maximum depth of water that can be infiltrated within the required drain time (ft)

P_{design} = design percolation rate of underlying soils (in/hr)

t = required drain time (hrs)

2) Choose the ponding depth (d_p) and/or trench depth (d_t) such that:

$d_{\max} \geq d_p$ For Infiltration Basins (Equation E-13)

$d_{\max} \geq n_t d_t + d_p$ For Infiltration Trenches or aggregate-filled Drywells (Equation E-14)

Where:

d_{max} = the maximum depth of water that can be infiltrated within the required drain time (ft)

d_p = ponding depth (ft)

n_t = trench/drywell fill aggregate porosity (unitless)

d_t = depth of trench/drywell filter media (ft)

3) Calculate infiltrating surface area (filter bottom area) required:

$A = \dfrac{SQDV}{((TP_{design}/12) + d_p)}$ For Infiltration Basins (Equation E-15)

$A = \dfrac{SQDV}{((TP_{design}/12) + n_t d_t + d_p)}$ For Infiltration Trenches or aggregate-filled Drywells (Equation E-16)

Where:

$SQDV$ = stormwater quality design volume (ft^3)

n_t = trench fill aggregate porosity (unitless)

P_{design} = design percolation rate (in/hr)

d_p = ponding depth (ft)

d_t = depth of trench filter media (ft)

T = fill time (time to fill to max ponding depth with water) (hrs) [use 2 hours for most designs]

Step 4: Size the forebay (applies to infiltration basins and trenches)

Infiltration facilities require pre-treatment to reduce sediment load into the basin. If a separate pre-treatment unit is not used, a forebay should be constructed for the facility. If a forebay is used, all inlets must enter the sediment forebay. The sediment forebay must be sized to 25% of the basin volume. The forebay must have interior slopes no steeper than 4:1.

1) Calculate the volume of the sediment forebay:

$$V_{forebay} = 0.25 \times SQDV \qquad \text{(Equation E-17)}$$

Where:

$V_{forebay}$ = Volume of sediment forebay

SQDV = Stormwater Quality Design Volume of Infiltration Basin

2) Select the depth of forebay, $d_{forebay}$. This is recommended to be...

3) Determine bottom surface area of forebay:

$$A_{forebay} = \frac{V_{forebay}}{d_{forebay}} \qquad \text{(Equation E-18)}$$

Where:

$A_{forebay}$ = Bottom surface area of forebay

$V_{forebay}$ = Volume of forebay

$d_{forebay}$ = Depth of forebay

4) Size forebay outlet pipe. Pipe must 8 inches in diameter, minimum, and must be sized such that the forebay drains completely within 10 minutes.

Step 5: Provide conveyance capacity for filter clogging

The infiltration facility should be placed off-line, but an emergency overflow must still be provided in the event the filter becomes clogged. Spillway and overflow structures should be designed in accordance with applicable standards of the Ventura County Flood Control District or local jurisdiction.

Sizing Worksheet

Step 1: Determine water quality design volume		
1-1. Enter Project area (acres), $A_{project}$	$A_{project}$ =	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (%) (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	%
1-3. Determine the maximum allowable effective impervious area (acres), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA=	acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p (1-imp)$	C =	
1-9. Enter design rainfall depth of the storm (in), P_i	P_i =	in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P =	ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560\times C*P*A_{retain}$	SQDV=	ft^3
Step 2: Determine the design percolation rate		
2-1. Enter measured soil percolation rate (in/hr, 0.5 in/hr min.), $P_{measured}$	$P_{measured}$ =	in/hr
2-2. Determine percolation rate correction factor, S_A based on suitability assessment (see Section 6 INF-1)	S_A =	

2-3. Determine percolation rate correction factor, S_B based on design (see Section 6 INF-1)	S_b =
2-4. Calculate combined safety factor, $S = S_A \times S_b$	S =
2-5. Calculate the design percolation rate (in/hr), $P_{design} = P_{measured}/S$	P_{design} = in/hr
Step 3: Calculate the surface area	
3-1. Enter required drain time(hours,72 hrs max.), t	t = hrs
3-2. Calculate max. depth of runoff that can be infiltrated within the t (ft), $d_{max} = P_{design}\ t/12$	d_{max} = ft
3-3. For basins, select ponding depth (ft), d_p, such that $d_p \leq d_{max}$	d_p = ft
3-4. For trenches, enter trench fill aggregate porosity, n_t	n_t =
3-5. For trenches, enter depth of trench fill (ft), d_t	d_t = ft
3-5. For trenches, select ponding depth d_p such that $d_p \leq d_{max} - n_t d_t$	d_p= ft
3-6. Enter the time to fill infiltration basin or trench with water (Use 2 hours for most designs), T	T = hrs
3-7. Calculate infiltrating surface area for infiltration basin (ft²): $A_b = SQDV/(T\ P_{design}\ /12+d_p)$ OR Calculate infiltrating surface area for infiltration trenches or aggregate- filled drywells (ft²): $A_t = SQDV/(T\ P_{design}\ /12+n_t d_t+dp)$	A_b = ft² A_t = ft²
Step 4: Size the forebay (infiltration basins or trenches)	
If a separate pre-treatment unit is designed for the infiltration facility, skip to Step 5. If not, continue through 4-1 through 4-4.	

4-1. Calculate the volume of the forebay (ft^3), $V_{forebay}=0.25*SQDV$	$V_{forebay}=$ ft^3
4-2. Determine forebay depth (ft), $d_{forebay}$	$d_{forebay}=$ ft
4-3. Calculate forebay bottom surface area (ft^2), $A_{forebay}=V_{forebay}/d_{forebay}$	$A_{forebay}=$ ft^2
4-4. Provide outlet pipe such that the forebay drains to the infiltration facility within 10 minutes.	
Step 5: Provide conveyance capacity for filter clogging	
5-1.The infiltration facility should be placed off-line, but an emergency overflow must still be provided in the event the filter becomes clogged. Design emergency overflow in accordance with applicable standards of the Ventura County Flood Control District or local jurisdiction.	

Design Example

Step 1: Determine water quality design volume

For this design example, a 10-acre residential development with a 60% total impervious area is considered to drain to an infiltration basin. The 85th percentile storm event for the project location is 0.75 inches.

Step 1: Determine water quality design volume			
1-1. Enter Project area (acres), $A_{project}$	A =	10	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (%) (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	5	
1-3. Determine the maximum allowable effective impervious area (acres), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	0.5	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	0.6	
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA=	6	acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	5.5	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	0.05	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\,(1-imp)$	C =	0.59	
1-9. Enter design rainfall depth of the storm (in), P_i	P_i =	0.75	in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P =	0.06	ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560\times C*P*A_{retain}$	SQDV =	8,500	ft^3

Step 2: Calculate Design Infiltration Rate

Infiltration facilities require a minimum soil infiltration rate of 0.5 in/hr. If the rate exceeds 2.4 in/hr as in this example, then the runoff should be fully treated in an upstream BMP prior to infiltration to protect the groundwater quality.

Step 2: Determine the design percolation rate			
2-1. Enter measured soil percolation rate (0.5 in/hr min.), $P_{measured}$	$P_{measured}$ =	4.0	in/hr
2-2. Determine percolation rate correction factor, S_A, based on suitability assessment (see Section 6 INF-1)	S_A =	3	
2-3. Determine percolation rate correction factor, S_B, based on design (see Section 6 INF-1)	S_b =	3	
2-4. Calculate combined safety factor, $S = S_A \times S_b$	S =	9	
2-5. Calculate the design percolation rate, $P_{design} = P_{measured}/S$	P_{design} =	0.44	in/hr

Step 3: Determine Facility Size

The size of the infiltrating surface is determined by assuming the SQDV will fill the available ponding depth (plus the void spaces of the computed porosity (usually about 32%) of the gravel in the trench).

Step 3: Calculate the surface area			
3-1. Enter drawdown time (72 hrs max.), t_d	t =	72	hrs
3-2. Calculate max. depth of runoff that can be infiltrated within the t, $d_{max} = P_{design}\ t/12$	d_{max}=	2.4	ft
3-3. Enter trench fill aggregate porosity, n_t	n_t=	0.32	
3-4. Enter depth of trench fill, d_t	d_t =	4	ft
3-5. Select trench ponding depth d_p such that $d_p \leq d_{max} - n_t d_t$	d_p=	1.1	ft
3-6. Enter the time to fill infiltration basin or trench with water (Use 2 hours for most designs), T	T =	2	hrs

3-7. Calculate infiltrating surface area for infiltration basin: $A_b = SQDV/(T\ P_{design}\ /12+d_p)$	A_b = 7,250 ft²

Step 4: Size the Forebay

A sediment forebay will be provided for this example as there is no separate pre-treatment unit provided.

Step 4: Size the forebay	
4-1. Calculate the volume of the forebay, $V_{forebay}$=0.25*SQDV	$V_{forebay}$= 2,100 ft³
4-2. Determine forebay depth, $d_{forebay}$	$d_{forebay}$= 3 ft
4-3. Calculate forebay bottom surface area, $A_{forebay}=V_{forebay}/d_{forebay}$	$A_{forebay}$= 700 ft²
4-4. Provide outlet pipe such that the forebay drains to the infiltration facility within 10 minutes.	

Step 5: Provide Conveyance Capacity for Flows Higher than Qwq

The infiltration facility should be placed off-line, but an emergency overflow for flows greater than the peak design storm must still be provided in the event the filter becomes clogged. Design emergency overflow in accordance with applicable standards of the Ventura County Flood Control District or local jurisdiction.

E.3 INF-3 Bioretention

Sizing Methodology

Bioretention areas can be sized using one of two methods: a simple sizing method or a routing method. The simple sizing procedure is summarized below. Continuous simulation modeling, routing spreadsheets, and/or other forms of routing modeling that incorporate rainfall-runoff relationships and infiltrative (flow) capacities of bioretention may be used to size facilities. Alternative sizing methodologies should be prepared with good engineering practices. For the routing modeling method, refer to the Sand Filter design guidance (FILT-1). A bioretention sizing worksheet and example are provided in this appendix. Planter boxes are sized the same as bioretention areas with underdrains using parameters appropriate for planter boxes.

With either method, the runoff entering the facility must completely drain the ponding area within 48 hours, and runoff must be completely infiltrated within 96 hours. Bioretention is to be sized, with or without underdrains, such that the SQDV will fill the available ponding depth, the void spaces in the planting soil, and the optional gravel layer below the media.

Step 1: Determine the stormwater quality design volume (SQDV)

Bioretention areas should be sized to capture and treat the water quality design volume (see Section E.1).

Step 2: Determine the Design Percolation Rate

The percolation rate will decline between maintenance cycles as particulates accumulate in the infiltrative layer and the surface becomes occluded. Additionally, monitoring of actual facility performance has shown that the full-scale infiltration rate is far lower than the rate measured by small-scale testing. It is important that adequate conservatism is incorporated in the selection of design percolation rates. For infiltrating bioretention facilities, the design percolation rate discussed here is the percolation rate of the underlying soils, which will drive infiltration through the facility. See INF-3: Bioretention for guidance in developing design percolation rate correction factors.

Step 3: Calculate the bioretention surface area

1) Determine the maximum depth of surface ponding that can be infiltrated within the required surface drain time:

$$d_{\max} = \frac{P_{design} \times t_{ponding}}{12\frac{in}{ft}}$$

Where:

$t_{ponding}$ = required drain time of surface ponding (48 hrs)

P_{design} = design percolation rate of underlying soils (in/hr) (see Step 2, above)

d_{max} = the maximum depth of surface ponding water that can be infiltrated within the required drain time (ft)

2) Choose surface ponding depth (dp) such that:

$$d_p \leq d_{\max} \quad \text{(Equation E-19)}$$

Where:

d_p = selected surface ponding depth (ft)

d_{max} = the maximum depth of water that can be infiltrated within the required drain time (ft)

3) Choose thickness(es) of amended media and aggregate layer(s) and calculate total effective storage depth of the bioretention area as follows:

$$d_{effective} \leq d_p + n^*_{media} l_{media} + n_{gravel} l_{gravel} \quad \text{(Equation E-20)}$$

Where:

$d_{effective}$ = total equivalent depth of water stored in bioretention area (ft)

d_p = surface ponding depth (ft)

n^*_{media} = available porosity of amended soil media (ft/ft), approximately 0.25 ft/ft accounting for antecedent moisture conditions

l_{media} = thickness of amended soil media layer (ft)

n_{gravel} = porosity of optional gravel layer (ft/ft), approximately 0.30 ft/ft

l_{gravel} = thickness of optional gravel layer (ft)

4) Check that entire effective depth (surface plus subsurface storage) infiltrates in no greater than 96 hours as follows:

$$t_{total} = \frac{d_{effective}}{P_{design}} \times 12 \frac{in}{ft} \leq 96\ hr \quad \text{(Equation E-21)}$$

Where:

$d_{effective}$ = total equivalent depth of water stored in bioretention area (ft)

P_{design} = design percolation rate of underlying soils (in/hr) (see Step 2, above)

If $t_{total} > 96$ hrs, then reduce surface ponding depth and/or amended media thickness and/or gravel thickness and return to Step [A].

If $t_{total} \leq 96$ hrs, then proceed to Step [E].

5) Calculate required infiltrating surface area (filter bottom area):

$$A_{req} = \frac{SQDV}{d_{effective}} \quad \text{(Equation E-22)}$$

Where:

$SQDV$ = stormwater quality design volume (ft3)

Step 4: Calculate the bioretention total footprint

Calculate total footprint required by including a buffer for side slopes and freeboard; A_{req} is measured at the as the filter bottom area (toe of side slopes).

Sizing Worksheet

Step 1: Determine water quality design volume	
1-1. Enter Project area (acres), $A_{project}$	$A_{project}$ = acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (%) (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ = %
1-3. Determine the maximum allowable effective impervious area (acres), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$= acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA= acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} = acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p (1-imp)$	C =
1-9. Enter design rainfall depth of the storm (in), P_i	P_i = in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P = ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560\times C*P*A_{retain}$	SQDV= ft^3

Step 2: Determine the design percolation rate	
2-1. Enter measured soil percolation rate (in/hr) (0.5 in/hr minimum), $P_{measured}$	$P_{measured}$ = in/hr
2-2. Determine percolation rate correction factor, S_A based on suitability assessment (see Section 6 INF-3)	S_A =

2-3. Determine percolation rate correction factor, S_B based on design (see Section 6 INF-3)	S_B =	
2-4. Calculate combined safety factor, $S = S_A \times S_b$	S =	
2-5. Calculate the design percolation rate (in/hr), $P_{design} = P_{measured}/S$	P_{design} =	in/hr
Step 3: Calculate Bioretention Infiltrating surface area		
3-1. Enter water quality design volume (ft^3), *SQDV*	SQDV =	ft^3
3-2. Enter design percolation rate (in/hr), P_{design}	P_{design} =	in/hr
3.3 Enter the required drain time (48 hours), $t_{ponding}$	$t_{ponding}$ =	hours
3-3. Calculate the maximum depth of surface ponding that can be infiltrated within the required drain time (ft): $d_{max} = (P_{design} \times t_{ponding})/12$	d_{max} =	ft
3-4. Select surface ponding depth (ft), d_p, such that $d_p \leq d_{max}$	d_p =	ft
3-5. Select thickness of amended media (ft,2 feet minimum, 3 preferred), l_{media}	l_{media} =	ft
3-6. Enter porosity of amended media (roughly 25% or 0.25 ft/ft), n_{media}	n_{media}=	ft/ft
3-7. Select thickness of optional gravel layer (ft), l_{gravel}	l_{gravel} =	ft
3-8. Enter porosity of gravel (roughly 30% or 0.3 ft/ft), n_{gravel}	n_{gravel}=	ft/ft
3-9. Calculate the total effective storage depth of bioretention facility (ft): $d_{effective} \leq (d_p + n_{media}l_{media} + n_{gravel}l_{gravel})$	$d_{effective}$=	ft

3-10. Check that the entire effective depth infiltrates in required drainage time, 96 hours: $t_{total} = (d_{effective}/P_{design}) \times 12$ If t_{total} > 96 hours, reduce surface ponding depth and/or amended media thickness and/or gravel thickness and return to 3-4. If $t_{total} \leq$ 96 hours, proceed to 3-11.	t_{total} = hours
3-11. Calculate the required infiltrating surface area (ft^2): $A_{req} = SQDV/d_{effective}$	A_{req} = ft^2
Step 4: Calculate Bioretention Area Total Footprint	
4-1. Calculate total footprint required by including a buffer for side slopes and freeboard (ft^2) [A_{req} is measured at the as the filter bottom area (toe of side slopes)], A_{tot}	A_{tot} = ft^2

Design Example

Bioretention areas have several components that allow the pretreatment, spreading, filtration, collection and discharge of the incoming flows.

Step 1: Determine water quality design volume

For this design example, a 10-acre site with soil type 4 and 60% total impervious area is considered. The 85th percentile storm event for the project location is 0.75 inches.

Step 1: Determine water quality design volume			
1-1. Enter Project area (acres), $A_{project}$	$A_{project}$ =	10	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (%) (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	5	
1-3. Determine the maximum allowable effective impervious area (acres), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	0.5	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	0.6	
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA=	6	acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	5.5	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	0.05	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\,(1\text{-}imp)$	C =	0.59	
1-9. Enter design rainfall depth of the storm (in), P_i	P_i =	0.75	in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P =	0.06	ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560\times C*P*A_{retain}$	SQDV=	8,500	ft^3

Step 2: Determine the design percolation rate

For this design example, a native soil percolation rate of 1.5 in/hr is assumed.

Step 2: Determine the design percolation rate	
2-1. Enter measured soil percolation rate (in/hr, 0.5 in/hr minimum), $P_{measured}$	$P_{measured}$ = 4.0 in/hr
2-2. Determine percolation rate correction factor, S_A, based on suitability assessment (see Section 6 INF-1)	S_A = 3
2-3. Determine percolation rate correction factor, S_B, based on design (see Section 6 INF-1)	S_b = 3
2-4. Calculate combined safety factor, $S = S_A \ x\ S_b$	S = 9
2-5. Calculate the design percolation rate (in/hr), $P_{design} = P_{measured}/S$	P_{design} = 0.44 in/hr

Step 3: Determine bioretention/ planter box area footprint

A bioretention area is designed with two components: (1) temporary storage reservoir to store runoff, and (2) a plant mix filter bed (planting soil mixed with sand content = 70%) through which the stored runoff must percolate to obtain treatment.

Step 3: Calculate bioretention/planter box surface area	
3-1. Enter water quality design volume (ft^3), *SQDV*	SQDV = 8,500 ft^3
3-2. Enter design percolation rate (in/hr), P_{design}	P_{design} = 0.375 in/hr
3.3 Enter the required drain time (48 hours), $t_{ponding}$	$t_{ponding}$ = 48 hours
3-3. Calculate the maximum depth of surface ponding (ft) that can be infiltrated within the required drain time (48 hours): $d_{max} = (P_{design} \times t_{ponding})/12$	d_{max} = 1.5 ft
3-4. Select surface ponding depth d_p such that $d_p \leq d_{max}$	d_p = 1.5 ft
3-5. Select thickness of amended media (2 feet minimum, 3 preferred), l_{media}	l_{media} = 3 ft

Step 3: Calculate bioretention/planter box surface area	
3-6. Enter porosity of amended media (roughly 25% or 0.25 ft/ft), n_{media}	n_{media}= 0.25 ft/ft
3-7. Select thickness of optional gravel layer (ft), l_{gravel}	l_{gravel} = 1 ft
3-8. Enter porosity of gravel (roughly 30% or 0.3 ft/ft), n_{gravel}	n_{gravel}= 0.3 ft/ft
3-9. Calculate the total effective storage depth of bioretention facility (ft): $d_{effective} \leq (d_p + n_{media}l_{media} + n_{gravel}l_{gravel})$	$d_{effective}$= 2.6 ft
3-10. Check that the entire effective depth infiltrates in required drainage time, 96 hours: $t_{total} = (d_{effective}/P_{design}) \times 12$ If t_{total} > 96 hours, reduce surface ponding depth and/or amended media thickness and/or gravel thickness and return to 3-4. If $t_{total} \leq$ 96 hours, proceed to 3-11.	t_{total} = 82 hours
3-11. Calculate the required infiltrating surface area (ft²), $A_{req} = SQDV/d_{effective}$	A_{req} = 3,300 ft²

Step 4: Calculate Bioretention Area Total Footprint

For this design example, a natural-shaped bioretention area is assumed, with 3:1 side slopes. To calculate the total footprint, the side slopes would be added to the design geometry.

E.4 INF-5 Permeable Pavement

Sizing Methodology

Permeable pavement (including the base layers) shall be designed to drain in less than 72 hours. The basis for this is that soils must be allowed to dry out periodically in order to restore hydraulic capacity; this is essential in order to receive flows from subsequent storms, maintain infiltration rates, maintain adequate sub soil oxygen levels for healthy soil biota, and to provide proper soil conditions for biodegradation and retention of pollutants.

Permeable pavement must be built and designed by a licensed civil engineer in accordance with Ventura County roadway and pavement specifications.

Step 1: Calculate the design volume

Permeable pavement shall be sized to capture and treat the stormwater quality design volume, SQDV (see Section 2 and Appendix E).

Step 2: Determine the Design Percolation Rate

The percolation rate will decline between maintenance cycles as particulates accumulate in the infiltrative layer and the surface becomes occluded. Additionally, monitoring of actual facility performance has shown that the full-scale infiltration rate is far lower than the rate measured by small-scale testing. It is important that adequate conservatism is incorporated in the selection of design percolation rates. For infiltrating bioretention facilities, the design percolation rate discussed here is the percolation rate of the underlying soils, which will drive infiltration through the facility. See INF-5: Permeable Pavement for guidance in developing design percolation rate correction factors.

Step 3: Determine gravel drainage layer depth

Permeable pavement (including the base layers) shall be designed to drain in less than 72 hours. The basis for this is that soils must be allowed to dry out periodically in order to restore hydraulic capacity to receive flows from subsequent storms, maintain infiltration rates, maintain adequate sub soil oxygen levels for healthy soil biota, and to provide proper soil conditions for biodegradation and retention of pollutants.

1) Calculate the maximum depth of runoff, d_{max}, that can be infiltrated within the drawdown time:

$$d_{\max} = \frac{P_{design} \bullet t}{12} \qquad \text{(Equation E-23)}$$

Where:

d_{max} = maximum depth that can be infiltrated (ft)

P_{design} = design percolation rate of underlying soils (in/hr) (see Step 2, above)

t = drawdown time (72 hrs maximum) (hr)

1) Select the gravel drainage layer depth, l, such that:

$$d_{\max} \geq n \times l \qquad \text{(Equation E-24)}$$

Where:

d_{max} = maximum depth that can be infiltrated (ft) (see 1) above)

n = gravel drainage layer porosity(unitless) (generally about 32% or 0.32 for gravel)

l = gravel drainage layer depth (ft)

Step 4: Determine infiltrating surface area

1) Calculate infiltrating surface area for permeable pavement, A:

$$A = \frac{SQDV}{\frac{TP_{design}}{12} + nl} \qquad \text{(Equation E-25)}$$

Where:

P_{design} = design percolation rate of underlying soils (in/hr) (see Step 2, above)

n = gravel drainage layer porosity(unitless)[about 32% or 0.32 for gravel]

l = depth of gravel drainage layer (ft)

T = time to fill the gravel drainage layer with water (use 2 hours for most designs) (hr)

Step 5: Provide conveyance capacity for clogging

The permeable pavement must have an emergency overflow for storm events greater than the design and in the event the permeable pavement becomes clogged. See INF-5 Permeable Pavement for overflow details.

Sizing Worksheet

Step 1: Determine water quality design volume	
1-1. Enter Project area (acres), $A_{project}$	$A_{project}$ = acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (%) (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ %
1-3. Determine the maximum allowable effective impervious area (acres), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$= acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA= acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} = acres
1-7. Determine pervious runoff coefficient using <u>Table E-1</u>, C_p	C_p =
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p (1-imp)$	C =
1-9. Enter design rainfall depth of the storm (in), P_i	P_i = in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P = ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560 \times C*P*A_{retain}$	SQDV= ft^3
Step 2: Determine the design percolation rate	
2-1. Enter measured soil percolation rate (0.5 in/hr minimum), $P_{measured}$	$P_{measured}$ = in/hr
2-2. Determine percolation rate correction factor, S_A based on suitability assessment (see Section 6 INF-5)	S_A =

Step 2: Determine the design percolation rate	
2-3. Determine percolation rate correction factor, S_B based on design (see Section 6 INF-5)	S_B =
2-4. Calculate combined safety factor, $S = S_A \times S_b$	S =
2-5. Calculate the design percolation rate (in/hr), $P_{design} = P_{measured}/S$	P_{design} = in/hr
Step 3: Determine the Gravel Drainage Layer Depth	
3-1. Enter drawdown time (hours, 72 hrs max.), t	t = hours
3-2. Calculate max. depth of runoff (ft) that can be infiltrated within the t, $d_{max}=P_{design}t/12$	d_{max} = ft
3-3. Enter the gravel drainage layer porosity, n (typically 32% or 0.32 for gravel)	n =
3-4. Select the gravel drainage layer depth (ft) such that $d_{max} \geq n \times l$	l = ft
Step 4: Determine infiltrating surface area	
4-1. Enter gravel drainage layer porosity, n	n =
4-2. Enter depth of gravel drainage layer (ft), l	l = ft
4-3. Enter the time to fill the gravel drainage layer with water (Use 2 hours for most designs), T	T = hrs
4-4. Calculate infiltrating surface area (ft^3): $A=SQDV/((TP_{design}/12)+nl)$	A = ft^2
Step 5: Provide conveyance capacity for clogging	
5-1. The permeable pavement must have an emergency overflow for storm events greater than the design and in the event the permeable pavement becomes clogged.	

Design Example

Step 1: Determine Water Quality Design Volume

For this design example, a 10-acre residential development with a 60% total impervious area is considered. The 85th percentile storm event for the project location is 0.75 inches.

Step 1: Determine Water Quality Design Volume			
1-1. Enter Project area (acres), $A_{project}$	A =	10	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (%) (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowableble}$ =	5	
1-3. Determine the maximum allowable effective impervious area (acres), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	0.5	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	0.6	
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA=	6	acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	5.5	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	0.05	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\ (1-imp)$	C =	0.59	
1-9. Enter design rainfall depth of the storm (in), P_i	P_i =	0.75	in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P =	0.06	ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560\times C*P*A_{retain}$	SQDV =	8,500	ft^3

Step 2: Calculate Design Percolation Rate

Permeable pavement with no underdrain requires a minimum soil infiltration rate of 0.5 in/hr. For this design example, a native soil percolation rate of 1.5 in/hr is assumed.

Step 2: Determine the design percolation rate			
2-1. Enter measured soil percolation rate (0.5 in/hr min.), $P_{measured}$	$P_{measured}$ =	4.0	in/hr
2-2. Determine percolation rate correction factor, S_A, based on suitability assessment (see Section 6 INF-1)	S_A =	3	
2-3. Determine percolation rate correction factor, S_B, based on design (see Section 6 INF-1)	S_b =	3	
2-4. Calculate combined safety factor, $S = S_A \ x \ S_b$	S =	9	
2-5. Calculate the design percolation rate (in/hr), $P_{design} = P_{measured}/S$	P_{design} =	0.44	in/hr

Step 3: Determine maximum depth that can be infiltrated

Based on the design infiltration rate and the max drawdown, determine the maximum depth that can be infiltrated within the time constraints.

Step 3: Determine maximum depth that can be infiltrated			
3-1. Enter drawdown time (72 hrs max.), t	t =	72	hrs
3-2. Calculate max. depth of runoff (ft) that can be infiltrated within the t, $d_{max}=P_{design}t/12$	d_{max} =	2.6	ft
3-3. Enter the gravel drainage layer porosity, n (typically 32% or 0.32 for gravel)	n =	0.32	
3-4. Select the gravel drainage layer depth (ft) such that $d_{max} \geq n \times l$	l =	8	ft

Step 4: Determine the infiltrating surface area (pavement area)

Using the depth calculated in Step 3, the required infiltrating surface area of the pavement can be calculated.

Step 4: Determine the infiltrating surface area	
4-1. Enter gravel drainage layer porosity, *n*	n = 0.32
4-2. Enter depth of gravel drainage layer (ft), *l*	l = 8 ft
4-3. Enter the time to fill the gravel drainage layer with water (Use 2 hours for most designs), *T*	T = 2 hrs
4-4. Calculate infiltrating surface area (ft^3): $A=SQDV/(TP_{design}/12)+n*l))$	A = 1,630 ft^2

Step 5: Provide conveyance capacity for clogging

The permeable pavement must have an emergency overflow for storm events greater than the design and in the event the permeable pavement becomes clogged.

E.5 VEG-1 Bioretention/VEG-2 Planter Box

Sizing Methodology

Bioretention areas can be sized using one of two methods: a simple sizing method or a routing method. The simple sizing procedure is summarized below. Continuous simulation modeling, routing spreadsheets, and/or other forms of routing modeling that incorporate rainfall-runoff relationships and infiltrative (flow) capacities of bioretention may be used to size facilities. Alternative sizing methodologies should be prepared with good engineering practices. For the routing modeling method, refer to the Sand Filter design guidance (FILT-1). A bioretention sizing worksheet and example are provided in this appendix. Planter boxes are sized the same as bioretention areas with underdrains using parameters appropriate for planter boxes.

With either method, the runoff entering the facility must completely drain the ponding area within 48 hours, and runoff must be completely infiltrated within 96 hours. Bioretention is to be sized, with or without underdrains, such that the SQDV will fill the available ponding depth, the void spaces in the planting soil, and the optional aggregate layer.

Step 1: Determine the stormwater quality design volume (SQDV)

Bioretention areas should be sized to capture and treat the water quality design volume (see Section E.1).

Step 2: Determine the Design Percolation Rate

Sizing is based on the design saturated hydraulic conductivity (K_{sat}) of the amended soil layer. A target K_{sat} of 5 inches per hour is recommended for newly installed non-proprietary amended soil media. The media K_{sat} will decline between maintenance cycles as the surface becomes occluded and particulates accumulate in the amended soil layer. A factor of safety of 2.0 should be applied such that the resulting recommended design percolation rate is 2.5 inches per hour. This value should be used for sizing unless sufficient rationale is provided to justify a higher design percolation rate.

Step 3: Calculate the bioretention or planter box surface area

Determine the size of the required infiltrating surface by assuming the SQDV will fill the available ponding depth plus the void spaces in the media, based on the computed porosity of the filter media and optional aggregate layer.

1) Select a surface ponding depth (d_p) that satisfies geometric criteria and congruent with the constraints of the site. Selecting a deeper ponding depth (18 inches maximum) generally yields a smaller footprint, however requires greater consideration for public safety and energy dissipation.

2) Compute time for selected ponding depth to filter through media:

$$t_{ponding} = \frac{d_p}{K_{design}} 12 \frac{in}{ft} \leq 48 \text{ hours} \qquad \text{(Equation E-26)}$$

Where:

$t_{ponding}$ = required drain time of surface ponding (48 hrs)

d_p = selected surface ponding water depth (ft)

K_{design} = design saturated hydraulic conductivity (in/hr) (see Step 2, above)

If $t_{ponding}$ exceeds 48 hours, return to (1) and reduce surface ponding or increase media K_{design}. Otherwise, proceed to next step.

Note: In nearly all cases, $t_{ponding}$ will not approach 48 hours unless a low Kdesign is specified.

3) Compute depth of water that may be considered to be filtered during the design storm event as follows:

$$d_{filtered} = Minimum\left[\frac{K_{design} \times T_{routing}}{12 \, {in}/{ft}}, \frac{d_p}{2}\right] \qquad \text{(Equation E-27),}$$

Where:

$d_{filtered}$ = depth of water that may be considered to be filtered during the design storm event (ft) for routing calculations; this value should not exceed half of the surface ponding depth (d_p)

K_{design} = design saturated hydraulic conductivity (in/hr) (see Step 2, above)

$T_{routing}$ = storm duration that may be assumed for routing calculations; this should be assumed to be **3 hours** unless rationale for an alternative assumption is provided

d_p = selected surface ponding water depth (ft)

4) Calculate required infiltrating surface area (filter bottom area):

$$A_{req} = \frac{SQDV}{d_p + d_{filtered}} \qquad \text{(Equation E-28)}$$

Where:

A_{req}	=	required area at bottom of filter area (ft^2); does not account for side slopes and freeboard
$SQDV$	=	stormwater quality design volume (ft^3)
d_p	=	selected surface ponding water depth (ft)
$d_{filtered}$	=	depth of water that can be considered to be filtered during the design storm event (ft) for routing calculations (See previous step)

Step 4: Calculate the bioretention total footprint

Calculate total footprint required by including a buffer for side slopes and freeboard; A_{req} is measured at the filter bottom area (toe of side slopes).

Step 5: Calculate underdrain system capacity

Underdrains are required for planter boxes and bioretention with underdrains. For guidance on sizing, refer to step 5 of the worksheet below. Alternatively, the Ventura County Hydrology Manual can be used for pipe sizing guidance.

Sizing Worksheet

Step 1: Determine water quality design volume		
1-1. Enter Project area (acres), $A_{project}$	$A_{project}$ =	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$	%
1-3. Determine the maximum allowed effective impervious area (ac), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA=	acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	acres
1-7. Determine pervious runoff coefficient using <u>Table E-1</u>, C_p	C_p =	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\ (1-imp)$	C =	
1-9. Enter design rainfall depth of the storm (in), P_i	P_i =	in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P =	ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560\bullet C*P*A_{retain}$	SQDV=	ft^3

Step 2: Determine the design percolation rate		
2-1. Enter the design saturated hydraulic conductivity of the amended filter media (2.5 in/hr recommended rate), K_{design}	K_{design} =	in/hr

Step 3: Calculate Bioretention/Planter Box surface area		
3-1. Enter water quality design volume (ft^3), *SQDV*	SQDV =	ft^3
3-2. Enter design saturated hydraulic conductivity (in/hr), K_{design}	K_{design} =	in/hr
3-3. Enter ponding depth (max 1.5 ft for Bioretention, 1 ft for Planter Box) above area, d_p	d_p =	ft
3-4. Calculate the drawdown time for the ponded water to filter through media (hours), $t_{ponding} = (d_p/K_{design}) \times 12$	$t_{ponding}$=	hrs
3-5. Enter the storm duration for routing calculations (use 3 hours unless there is rationale for an alternative), $T_{routing}$	$T_{routing}$ =	hrs
3-6. Calculate depth of water (ft) filtered by using the following two equations: $d_{filtered,1} = (K_{design} \times T_{routing})/12$ $d_{filteret,2} = d_p /2$	$d_{filtered,1}$ = $d_{filtered,2}$ =	ft ft
3.7 Enter the resultant depth (ft) (the lesser of the two calculated above), $d_{filtered}$	$d_{filtered}$ =	ft
3-8. Calculate the infiltrating surface area as follows (ft^2): $A_{req} = SQDV/(d_p + d_{filtered})$	A_{req} =	ft^2
Step 4: Calculate Bioretention Area Total Footprint		
4-1. Calculate total footprint required by including a buffer for side slopes and freeboard (ft^2) [A_{req} is measured at the as the filter bottom area (toe of side slopes)], A_{tot}	A_{tot} =	ft^2
Step 5: Calculate Underdrain System Capacity		
To calculate the underdrain system capacity, continue through steps 5-1 to 5-7.		

Step 5: Calculate Underdrain System Capacity		
5-1. Calculated filtered flow rate to be conveyed by the longitudinal drain pipe, $Q_f = K_{design}\ A_{req}/43{,}200$	Q_f =	cfs
5-2. Enter minimum slope for energy gradient, S_e	S_e =	
5-3. Enter Hazen-Williams coefficient for plastic, C_{HW}	C_{HW} =	
5-4. Enter pipe diameter (min 6 inches), D	D =	in
5-5. Calculate pipe hydraulic radius (ft), $R_h = D/48$	R_h =	ft
5-6. Calculate velocity at the outlet of the pipe (ft/s), $V_p = 1.318 C_{HW} R_h^{0.63} S_e^{0.54}$	V_p =	ft/s
5-7. Calculate pipe capacity (cfs), $Q_{cap} = 0.25\pi(D/12)^2 V_p$	Q_{cap} =	cfs

Design Example

Bioretention areas have several components that allow the pretreatment, spreading, filtration, collection and discharge of the incoming flows.

Step 1: Determine water quality design volume

For this design example, a 10-acre residential development with a 60% total impervious area is considered. The 85th percentile storm event for the project location is 0.75 inches.

Step 1: Determine Water Quality Design Volume			
1-1. Enter drainage area, A	A =	10	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowableble}$ =	5	
1-3. Determine the maximum allowed effective impervious area, $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	0.5	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	0.6	
1-5. Determine the Project Total Impervious area, $TIA = A_{project}*Imp$	TIA=	6	acres
1-6. Determine the total area from which runoff must be retained, $A_{retain} = TIA - EIA_{allowable}$	A_{retain} =	5.5	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	0.05	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\,(1-imp)$	C =	0.59	
1-9. Enter design rainfall depth of the storm, P_i (in)	P_i =	0.75	in
1-10. Calculate rainfall depth, $P = P_i/12$	P =	0.06	ft
1-11. Calculate water quality design volume, $SQDV = 43560 \cdot P*A_{retain}*C$	SQDV =	8,500	ft^3

Step 2: Determine the design percolation rate

For this design example, the recommended amended filter hydraulic conductivity is used, 2.5 in/hr.

Step 2: Determine the design percolation rate	
2-1. Enter the design saturated hydraulic conductivity of the amended filter media (2.5 in/hr recommended rate), K_{design}	K_{design} = 2.5 in/hr

Step 3: Determine bioretention/ planter box area footprint

A bioretention area is designed with two components: (1) temporary storage reservoir to store runoff, and (2) a plant mix filter bed (planting soil mixed with sand content = 70%) through which the stored runoff must percolate to obtain treatment.

Step 3: Calculate Bioretention/Planter Box surface area			
3-1. Enter water quality design volume (ft^3), *SQDV*	SQDV =	8,500	ac-ft
3-2. Enter design saturated hydraulic conductivity (in/hr), K_{design}	K_{design} =	2.5	in/hr
3-3. Enter ponding depth (max 1.5 ft for Bioretention, 1 ft for Planter Box) above area, d_p	d_p =	1.5	ft
3-4. Calculate the drawdown time for the ponded water to filter through media (hours), $t_{ponding} = (d_p/K_{design}) \times 12$	$t_{ponding}$=	7.2	hrs
3-5. Enter the storm duration for routing calculations (use 3 hours unless there is rationale for an alternative), $T_{routing}$	$T_{routing}$ =	3	hrs
3-6. Calculate depth of water (ft) filtered by using the minimum of the following two equations: $d_{filtered,1} = (K_{design} \times T_{routing})/12$ $d_{filteret,2} = d_p\ /2$	$d_{filtered,1}$ = $d_{filtered,2}$ =	0.63 0.75	ft ft
3.7 Enter the resultant depth (the minimum of the two calculated above), $d_{filtered}$	$d_{filtered}$ =	0.63	ft
3-8. Calculate the infiltrating surface area as follows (ft^2): $A_{req} = SQDV/(d_p + d_{filtered})$	A_{req} =	4,000	ft^2

Step 4: Calculate Bioretention Area Total Footprint

For this design example, a natural-shaped bioretention area is assumed, with 3:1 side slopes. To calculate the total footprint, the side slopes would be added to the design geometry.

Step 5: Calculate filter longitudinal underdrain collection pipe

All underdrain pipes must be 6 inches or greater in diameter to facilitate cleaning.

Step 5: Calculate underdrain system (required for planter box)			
To calculate the underdrain system capacity, continue through steps 5-1 to 5-7. If you don't need to calculate the underdrain capacity, skip this step.			
5-1. Calculated filtered flow rate to be conveyed by the longitudinal drain pipe (cfs), $Q_f = K_{design}\ A_{req}/43{,}200$	$Q_f =$	0.085	cfs
5-2. Enter minimum slope for energy gradient, S_e	$S_e =$	0.005	
5-3. Enter Hazen-Williams coefficient for plastic, C_{HW}	$C_{HW} =$	140	
5-4. Enter pipe diameter (min 6 in), D	$D =$	6	in
5-5. Calculate pipe hydraulic radius (ft), $R_h = D/48$	$R_h =$	0.13	ft
5-6. Calculate velocity at the outlet of the pipe (ft/s), $V_p = 1.318 C_{HW} R_h^{0.63} S_e^{0.54}$	$V_p =$	2.9	ft/s
5-7. Calculate pipe capacity (cfs), $Q_{cap} = 0.25\pi(D/12)^2 V_p$	$Q_{cap} =$	0.57	cfs

E.6 VEG-3 Vegetated Swale

Sizing Methodology

The flow capacity of a vegetated swale is a function of the longitudinal slope (parallel to flow), the resistance to flow (i.e. Manning's roughness), and the cross sectional area. The cross section is normally approximately trapezoidal and the area is a function of the bottom width and side slopes. The flow capacity of vegetated swales should be such that the design water quality flow rate will not exceed a flow depth of 2/3 the height of the vegetation within the swale or 4 inches at the water quality design flow rate. Once design criteria have been selected, the resulting flow depth for the design water quality design flow rate is checked. If the depth restriction is exceeded, swale parameters (e.g. longitudinal slope, width) are adjusted to reduce the flow depth.

Procedures for sizing vegetated swales are summarized below. A vegetated swale sizing worksheet and example are also provided.

Step 1: Select design flows

The swale sizing is based on the stormwater quality design flow SQDF (see Section E.1).

Step 2: Calculate swale bottom width

The swale bottom width is calculated based on Manning's equation for open-channel flow. This equation can be used to calculate discharges as follows:

$$Q = \frac{1.49 A R^{0.67} S^{0.5}}{n}$$

(Equation E-29)

Where:

Q	=	flow rate (cfs)
n	=	Manning's roughness coefficient (unitless)
A	=	cross-sectional area of flow (ft²)
R	=	hydraulic radius (ft) = area divided by wetted perimeter
S	=	longitudinal slope (ft/ft)

For shallow flow depths in swales, channel side slopes are ignored in the calculation of bottom width. Use the following equation (a simplified form of Manning's formula) to estimate the swale bottom width:

$$b = \frac{SQDF * n_{wq}}{1.49 y^{0.67} s^{0.5}} \quad \text{(Equation E-30)}$$

Where:

b	=	bottom width of swale (ft)
$SQDF$	=	stormwater quality design flow (cfs)
n_{wq}	=	Manning's roughness coefficient for shallow flow conditions = 0.2 (unitless)
y	=	design flow depth (ft)
s	=	longitudinal slope (along direction of flow) (ft/ft)

Proceed to Step 3 if the bottom width is calculated to be between 2 and 10 feet. A minimum 2-foot bottom width is required. Therefore, if the calculated bottom width is less than 2 feet, increase the width to 2 feet and recalculate the design flow depth y using the Equation 4-13, where Q_{wq}, n_{wq}, and s are the same values as used above, but b = 2 feet.

The maximum allowable bottom width is 10 feet; therefore if the calculated bottom width exceeds 10 feet, then one of the following steps is necessary to reduce the design bottom width:

1) Increase the longitudinal slope (s) to a maximum of 6 feet in 100 feet (0.06 feet per foot).

2) Increase the design flow depth (y) to a maximum of 4 inches.

3) Place a divider lengthwise along the swale bottom (Figure 3-1) at least three-quarters of the swale length (beginning at the inlet), without compromising the design flow depth and swale lateral slope requirements. Swale width can be increased to an absolute maximum of 16 feet if a divider is provided.

Step 3: Determine design flow velocity

To calculate the design flow velocity through the swale, use the flow continuity equation:

$$V_{wq} = SQDF/A_{wq} \quad \text{(Equation E-31)}$$

Where:

V_{wq}	=	design flow velocity (fps)
$SQDF$	=	stormwater quality design flow (cfs)

A_{wq}	=	$by + Zy^2$ = cross-sectional area (ft^2) of flow at design depth, where Z = side slope length per unit height (e.g., Z = 3 if side slopes are 3H:1V)

If the design flow velocity exceeds 1 foot per second, go back to Step 2 and modify one or more of the design parameters (longitudinal slope, bottom width, or flow depth) to reduce the design flow velocity to 1 foot per second or less. If the design flow velocity is calculated to be less than 1 foot per second, proceed to Step 4. *Note: It is desirable to have the design velocity as low as possible, both to improve treatment effectiveness and to reduce swale length requirements.*

Step 4: Calculate swale length

Use the following equation to determine the necessary swale length to achieve a hydraulic residence time of at least 7 minutes:

$$L = 60 t_{hr} V_{wq} \qquad \text{(Equation E-32)}$$

Where:

L	=	minimum allowable swale length (ft)
t_{hr}	=	hydraulic residence time (min)
V_{wq}	=	design flow velocity (fps)

The minimum swale length is 100 feet; therefore, if the swale length is calculated to be less than 100 feet, increase the length to a minimum of 100 feet, leaving the bottom width unchanged. If a larger swale can be fitted on the site, consider using a greater length to increase the hydraulic residence time and improve the swale's pollutant removal capability. If the calculated length is too long for the site, or if it would cause layout problems, such as encroachment into shaded areas, proceed to Step 5 to further modify the layout. If the swale length can be accommodated on the site (meandering may help), proceed to Step 6.

Step 5: Adjust swale layout to fit on site

If the swale length calculated in Step 4 is too long for the site, the length can be reduced (to a minimum of 100 feet) by increasing the bottom width up to a maximum of 16 feet, as long as the 10 minute retention time is retained. However, the length cannot be increased in order to reduce the bottom width because Manning's depth-velocity-flow rate relationships would not be preserved. If the bottom width is increased to greater than 10 feet, a low flow dividing berm is needed to split the swale cross section in half to prevent channelization.

Length can be adjusted by calculating the top area of the swale and providing an equivalent top area with the adjusted dimensions.

1) Calculate the swale treatment top area based on the swale length calculated in Step 4:

$$A_{top} = (b_i + b_{slope})L_i \qquad \text{(Equation E-33)}$$

Where:

A_{top} = top area (ft^2) at the design treatment depth

b_i = bottom width (ft) calculated in Step 2

b_{slope} = the additional top width (ft) above the side slope for the design water depth (for 3:1 side slopes and a 4-inch water depth, b_{slope} = 2 feet)

L_i = initial length (ft) calculated in Step 4

2) Use the swale top area and a reduced swale length L_f to increase the bottom width, using the following equation:

$$L_f = A_{top} / (b_f + b_{slope}) \qquad \text{(Equation E-34)}$$

Where:

L_f = reduced swale length (ft)

b_f = increased bottom width (ft).

3) Recalculate V_{wq} according to Step 3 using the revised cross-sectional area A_{wq} based on the increased bottom width b_f. Revise the design as necessary if the design flow velocity exceeds 1 foot per second.

4) Recalculate to assure that the 10 minute retention time is retained.

Step 6: Provide conveyance capacity for flows higher than SQDF

Vegetated swales may be designed as flow-through channels that convey flows higher than the water quality design flow rate, or they may be designed to incorporate a high-flow bypass upstream of the swale inlet. A high-flow bypass usually results in a smaller swale size. If a high-flow bypass is provided, this step is not needed. If no high-flow bypass is provided, proceed with the procedure below. Flow splitter structure design is described in Appendix G.

1) Check the swale size to determine whether the swale can convey the flood control design storm peak flows (Refer to the Ventura County Hydrology Manual, 2006).

2) The peak flow velocity of the flood control design storm (e.g., flood control design storm – see Ventura County Hydrology Manual, 2006)) must be less than 3.0 feet per second. If this velocity exceeds 3.0 feet per second, return to Step 2 and

increase the bottom width or flatten the longitudinal slope as necessary to reduce the flood control design storm peak flow velocity to 3.0 feet per second or less. If the longitudinal slope is flattened, the swale bottom width must be recalculated (Step 2) and must meet all design criteria.

Sizing Worksheet

Step 1: Determine water quality design flow		
1-1. Enter Project area (acres), $A_{project}$	A_{design} =	acres
1-2. Enter impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp =	
1-3. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	
1-4. Calculate runoff coefficient, $C = 0.95*imp + C_p\,(1\text{-}imp)$	C =	
1-5. Enter design rainfall intensity (in/hr), *i*	i =	in/hr
1-6. Calculate water quality design flow (cfs), *SQDF= CiA*	SQDF =	cfs
Step 2: Calculate swale bottom width		
2-1. Enter water quality design flow (cfs), *SQDF*	SQDF =	cfs
2-2. Enter Manning's roughness coefficient for shallow flow conditions, $n_{wq} = 0.2$	n_{wq} =	
2-3. Calculate design flow depth (ft), *y*	y =	ft
2-4. Enter longitudinal slope (ft/ft) (along direction of flow), *s*	s =	ft/ft
2-5. Calculate bottom width of swale (ft), $b = (SQDF*n_{wq})/(1.49y^{0.67}s^{0.5})$	b =	ft
2-6. If *b* is between 2 and 10 feet, go to Step 3		
2-7. If b is less than 2 ft, assume b = 2 ft and recalculate flow depth, $y = ((SQDF*n_{wq})/(2.98\,s^{0.5}))^{1.49}$	y =	ft

2-8. If b is greater than 10 ft, one of the following design adjustments must be made (recalculate variables as necessary): • Increase the longitudinal slope to a maximum of 0.06 ft/ft. • Increase the design flow depth to a maximum of 4 in (0.33 ft). • Place a divider lengthwise along the swale bottom (Figure 3-1) at least three-quarters of the swale length (beginning at the inlet). Swale width can be increased to an absolute maximum of 16 feet if a divider is provided.	
Step 3: Determine design flow velocity	
3-1. Enter side slope length per unit height (H:V) (e.g. 3 if side slopes are 3H :1V), Z	Z =
3-2. Enter bottom width of swale (ft), b	b = ft
3-3. Enter design flow depth (ft), y	y = ft
3-4. Calculate the cross-sectional area of flow at design depth (ft^2), $A_{wq} = by + Zy^2$	A_{wq} = ft^2
3-5. Calculate design flow velocity (ft/s), $V_{wq} = SQDF/ A_{wq}$	V_{wq} = ft/s
3-6. If the design flow velocity exceeds 1 ft/s, go back to Step 2 and change one or more of the design parameters to reduce the design flow velocity. If design flow velocity is less than 1 ft/s, proceed to Step 4.	
Step 4: Calculate swale length	
4-1. Enter hydraulic residence time (minutes, minimum 7 min), t_{hr}	t_{hr} = min
4-2. Calculate swale length (ft), $L = 60t_{hr}V_{wq}$	L = ft

Step 4: Calculate swale length		
4-3. If L is too long for the site, proceed to Step 5 to adjust the swale layout If L is greater than 100 ft and will fit within the constraints of the site, skip to Step 6 If L is less than 100 ft, increase the length to a minimum of 100 ft, leaving the bottom width unchanged, and skip to Step 6		
Step 5: Adjust swale layout to fit within site constraints		
5-1. Enter the bottom width calculated in Step 2 (ft), $b_i = b$	b_i =	ft
5-2. Enter design flow depth (ft), y	y=	ft
5-3. Enter the swale side slope ratio (H:V), Z	Z =	ft:ft
5-4. Enter the additional top width above the side slope for the design water depth (ft), $b_{slope} = 2Zy$	b_{slope} =	ft
5-5. Enter the initial length calculated in Step 4 (ft), $Li = L$	L_i =	ft
5-6. Calculate the top area at the design treatment depth (ft^2), $A_{top} = (b_i + b_{slope}) \times L_i$	A_{top} =	ft^2
5-7. Choose a reduced swale length based on site constraints (ft), L_f	L_f =	ft
5-8. Calculate the increased bottom width (ft), $b_f = (A_{top}/L_f) - b_{slope}$	b_f =	ft
5-9. Recalculate the cross-sectional area of flow at design depth (ft2), $A_{wq,f} = b_f y + Zy^2$	$A_{wq,f}$ =	ft^2
5-10. Recalculate design flow velocity (ft/s), $V_{wq} = SQDF/ A_{wq}$ Revise design as necessary if design flow velocity exceeds 1 ft/s.	V_{wq} =	ft/s

5-11. Recalculate the hydraulic residence time (min), $t_{hr} = L_f/(60V_{wq})$ Ensure that t_{hr} is greater or equal to 10 minutes.	t_{hr} = min
5-12. When V_{wq} and t_{hr} are recalculated to meet requirements, proceed to Step 6.	
Step 6: Provide conveyance capacity for flows higher than SQDF (if swale is on-line)	
6-1. If the swale already includes a high-flow bypass to convey flows higher than the water quality design flow rate, skip this step and verify that all parameters meet design requirements to complete sizing	
6-2. If swale does not include a high-flow bypass, determine that the swale can convey flood control design storm peak flows. Calculate the capital peak flow velocity per Ventura County requirements (ft/s), V_p	V_p = ft/s
6-3. If V_p > 3.0 feet per second, return to Step 2 and increase the bottom width or flatten the longitudinal slope as necessary to reduce the flood control design storm peak flow velocity to 3.0 feet per second or less. If the longitudinal slope is flattened, the swale bottom width must be recalculated (Step 2) and must meet all design criteria.	

Design Example

Step 1: Determine water quality design Flow

For this design example, a 10-acre site with Type 4 soil and 60% total imperviousness is considered. Flow-based sizing Method 1 is assumed. Therefore, the design intensity is 0.2 in/hr.

Step 1: Determine water quality design flow	
1-1. Enter Project area (acres), $A_{project}$	A = 10 acres
1-2. Enter impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp = 0.60
1-3. Determine pervious runoff coefficient using Table E-1, C_p	C_p = 0.05
1-4. Calculate runoff coefficient, $C = 0.95*imp + C_p\,(1\text{-}imp)$	C = 0.59
1-5. Enter design rainfall intensity (in/hr), *i*	i = 0.2 in/hr
1-6. Calculate water quality design flow (cfs), *SQDF= CiA*	*SQDF*= 1.18 cfs

Step 2: Calculate Swale Bottom Width

The swale bottom width is calculated based on Manning's equation. The grass height in the swale will be maintained at 6-inches. The design flow depth is assumed to be 2/3 of the grass height, or 4 inches (0.33 ft). The default Manning's roughness coefficient is assumed appropriate for expected vegetation density and design depth. The slope was assumed to be 0.04.

Step 2: Calculate swale bottom width	
2-1. Enter water quality design flow (cfs), *SQDF*	SQDF = 1.18 cfs
2-2. Enter Manning's roughness coefficient for shallow flow conditions, n_{wq} = 0.2	n_{wq} = 0.2
2-3. Calculate design flow depth (ft), *y*	*y* = 0.33 ft
2-4. Enter longitudinal slope (along direction of flow) (ft/ft), *s*	s = 0.04 ft/ft
2-5. Calculate bottom width of swale (ft),	b = 5.0 ft

Step 2: Calculate swale bottom width	
$b = Q_{wq}n_{wq} / 1.49y^{0.67}s^{0.5}$	
2-6. If *b* is between 2 and 10 feet, go to Step 3	
2-7. If *b* is less than 2 ft, assume *b* = 2 ft and recalculate flow depth, $y = (Q_{wq}n_{wq} / 2.98s^{0.5})^{1.49}$	Not applicable
2-8. If *b* is greater than 10 ft, one of the following design adjustments must be made (and recalculate as necessary): Increase the longitudinal slope to a maximum of 0.06 ft/ft. Increase the design flow depth to a maximum of 4 in (0.33 ft). Place a divider lengthwise along the swale bottom (Figure 3-1) at least three-quarters of the swale length (beginning at the inlet). Swale width can be increased to an absolute maximum of 16 feet if a divider is provided.	Not applicable

Step 3: Determine Design Flow Velocity

For this design example, it is assumed the side slopes will be designed as 3H: 1V, so Z = 3.

Step 3: Determine design flow velocity	
3-1. Enter side slope length per unit height (H:V) (e.g. 3 if side slopes are 3H :1V), *Z*	Z = 3
3-2. Enter bottom width of swale (ft), *b*	b = 5.0 ft
3-3. Enter design flow depth (ft), *y*	y = 0.33 ft
3-4. Calculate the cross-sectional area of flow at design depth (ft²), $A_{wq} = by + Zy^2$	A_{wq} = 2.0 ft²
3-5. Calculate design flow velocity (ft/s), $V_{wq} = SQDF/ A_{wq}$	V_{wq} = 0.59 ft/s
3-6. If the design flow exceeds 1 ft/s, go back to Step 2 and change one or more of the design parameters to reduce the design flow velocity. If design flow velocity is less than 1 ft/s, proceed to Step 4.	

Step 4: Calculate Swale Length

Using the design flow velocity and a minimum residence time of 7 minutes, the length of the swale is calculated as follows. The swale length must be a minimum of 100 ft.

Step 4: Calculate swale length	
4-1. Enter hydraulic residence time (min 7 min), t_{hr} (min)	t_{hr} = 10 min
4-2. Calculate swale length, $L = 60t_{hr}V_{wq}$	L = 354 ft
4-3. If L is too long for the site, proceed to Step 5 to adjust the swale layout If L is greater than 100 ft and will fit within the constraints of the site, skip to Step 6 If L is less than 100 ft, increase the length to a minimum of 100 ft, leaving the bottom width unchanged, and skip to Step 6	Not Applicable

Site constraints only allow a swale length of 300 feet. Therefore proceed to Step 5 to adjust the swale length.

Step 5: Adjust Swale Layout to Fit Within Site Constraints

To adjust swale length to 300 feet, the bottom width needs to be increased (up to a maximum of 16 ft if a divider is provided).

Step 5: Adjust swale layout to fit within site constraints	
5-1. Enter the bottom width calculated in Step 2 (ft), $b_i = b$	b_i = 5.0 ft
5-2. Enter design flow depth (ft), y	y= 0.33 ft
5-3. Enter the swale side slope ratio (H:V), Z	Z = 3 ft:ft
5-4. Enter the additional top width above the side slope for the design water depth (ft), $b_{slope} = 2Zy$	b_{slope} = 2 ft
5-5. Enter the initial length calculated in Step 4 (ft), $Li = L$	L_i = 354 ft
5-6. Calculate the top area at the design treatment depth (ft^2), $A_{top} = (b_i + b_{slope}) \times L_i$	A_{top} = 2,480 ft^2

5-7. Choose a reduced swale length based on site constraints (ft), L_f	L_f = 300 ft
5-8. Calculate the increased bottom width (ft), $b_f = (A_{top}/L_f) - b_{slope}$	b_f = 6.3 ft
5-9. Recalculate the cross-sectional area of flow at design depth (ft2), $A_{wq,f} = b_f y + Zy^2$	$A_{wq,f}$ = 2.4 ft^2
5-10. Recalculate design flow velocity (ft/s), $V_{wq} = SQDF/ A_{wq}$ Revise design as necessary if design flow velocity exceeds 1 ft/s.	V_{wq} = 0.49 ft/s
5-11. Recalculate the hydraulic residence time (min), $t_{hr} = L_f/(60V_{wq})$ Ensure that t_{hr} is greater or equal to 10 minutes.	t_{hr} = 10.2 min
5-12. When V_{wq} and t_{hr} are recalculated to meet requirements, proceed to Step 6.	

Since the new length and width yields V_{wq} and t_{hr} which meet requirements, continue to Step 6.

Step 6: Provide Conveyance Capacity for Flows Higher than SQDF

The swale will be offline such that all flows greater than SQDF will be bypassed.

E.7 VEG-4 Filter Strip

Sizing Methodology

The flow capacity of a vegetated filter strips (filter strips) is a function of the longitudinal slope (parallel to flow), the resistance to flow (e.g., Manning's roughness), and the width and length of the filter strip. The slope shall be small enough to ensure that the depth of water will not exceed 1 inch over the filter strip. Similarly, the flow velocity shall be less than 1 ft/sec. Procedures for sizing filter strips are summarized below. A filter strip sizing example is also provided.

Step 1: Calculate the design flow rate

The design flow is calculated based on the stormwater quality design flow rate, SQDF, as described in Section E.1.

Step 2: Calculate the minimum width

Determine the minimum width (i.e. perpendicular to flow) allowable for the filter strip and design for that width or larger.

$$W_{min} = (SQDF) / (q_{a,min}) \qquad \text{(Equation E-35)}$$

Where

W_{min} = minimum width of filter strip

$SQDF$ = stormwater quality design flow (cfs)

$q_{a,min}$ = minimum linear unit application rate, 0.005 cfs/ft

Step 3: Calculate the design flow depth

The design flow depth (d_f) is calculated based on the width and the slope (parallel to the flow path) using a modified Manning's equation as follows:

$$d_f = 12 * [SQDF * n_{wq} / 1.49 W_{trib} s^{0.5}]^{0.6} \qquad \text{(Equation E-36)}$$

Where:

d_f = design flow depth (inches)

$SQDF$ = stormwater quality design flow (cfs)

W_{trib} = width (perpendicular to flow = width of impervious surface contributing area (ft))

s = slope (ft/ft) of strip parallel to flow, average over the whole width

n_{wq}	=	Manning's roughness coefficient (0.25-0.30)

If d_f is greater than 1 inch (0.083 ft), then a shallower slope is required, or a filter strip cannot be used.

Step 4: Calculate the design velocity

The design flow velocity is based on the design flow, design flow depth, and width of the strip:

$$V_{wq} = SQDF / (d_f \, W_{trib}) \qquad \text{(Equation E-37)}$$

Where:

$d_{f,ft}$	=	design flow depth (ft) (d_f/12)
$SQDF$	=	stormwater quality design flow (cfs)
W_{trib}	=	width (perpendicular to flow = width of impervious surface contributing area (ft))

Step 5: Calculate the desired length of the filter strip

Determine the required length (L) to achieve a desired minimum residence time of 7 minutes using:

$$L = 60 t_{hr} V_{wq} \qquad \text{(Equation E-38)}$$

Where:

L	=	minimum allowable strip length (ft)
t_{hr}	=	hydraulic residence time (s)
V_{wq}	=	design flow velocity (fps)

Sizing Worksheet

Step 1: Calculate the design flow		
1-1. Enter Project area (acres), $A_{project}$	A_{design} =	acres
1-2. Enter impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp =	
1-3. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	
1-4. Calculate runoff coefficient, $C = 0.95*imp + C_p\,(1\text{-}imp)$	C =	
1-5. Enter design rainfall intensity (in/hr), *i*	i =	in/hr
1-6. Calculate water quality design flow (cfs), *SQDF= CiA*	SQDF =	cfs
Step 2: Calculate the minimum width		
2-1. Enter the stormwater quality design flow (cfs), *SQDF*	SQDF =	cfs
2-2. Enter the minimum linear unit application rate (0.005 cfs/ft), $q_{a,min}$	$q_{a,min}$=	cfs/ft
2-3. Calculate the minimum width of filter strip (ft), W_{min}	W_{min}=	ft
Step 3: Calculate the design flow depth		
3-1. Enter filter strip longitudinal slope, *s* (ft/ft)	*s* =	ft/ft
3-2. Enter Manning roughness coefficient (0.25-0.30), n_{wq}	n_{wq} =	
3-3. Enter width of impervious surface contributing area (perpendicular to flow), *W* (ft)	W =	ft

Step 3: Calculate the design flow depth	
3-4. Calculate average depth of water using Manning equation (inches), $d_f = 12 * [SQDF * n_{wq} / 1.49 W_{trib} s^{0.5}]^{0.6}$	d_f = inches
3-5. If d_f > 1" (0.083 ft), go back step 3-1 and decrease the slope	
3-6. If the slope cannot be changed due to construction constraints, go to step 3-3 and increase the width perpendicular to flow.	
Step 4: Calculate the design velocity	
4-1. Enter depth of water (ft), $d_{f,ft} = d_f / 12$	d_f = ft
4-2. Enter width of strip (ft), W	W = ft
4-3. Calculate design flow velocity (ft/s), $V_{wq} = SQDF/(d_{f,ft} W)$	V_{wq}= ft/s
4-4. If the V_{wq} >1 ft/s, go back to step 3-1 and decrease the slope.	
Step 5: Calculate the length of the filter strip	
5-1. Enter desired residence time (minimum 7 minutes), t	t = min
5-2. Enter design flow velocity (ft/s), V_{wq}	V_{wq}= ft/s
5-3. Calculate length of the filter strip (ft), $L = 60 t V_{wq}$	L = ft
5-4. If L < 4 ft, go to step 3-1 and increase the slope	

Design Example

Step 1: Determine water quality design Flow

For this design example, a 10-acre site with Type 4 soil and 60% total imperviousness is considered. Flow-based sizing Method 1 is used, as described in Section E.1.

Step 1: Calculate the design flow	
1-1. Enter Project area (acres), $A_{project}$	A_{design} = 10 acres
1-2. Enter impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp = 0.60
1-3. Determine pervious runoff coefficient using Table E-1, C_p	C_p = 0.05
1-4. Calculate runoff coefficient, $C = 0.95*imp + C_p\ (1\text{-}imp)$	C = 0.59
1-5. Enter design rainfall intensity (in/hr), *i*	i = 0.2 in/hr
1-6. Calculate water quality design flow (cfs), *SQDF= CiA*	SQDF = 1.18 cfs

Step 2: Calculate the minimum width of filter strip

Determine the minimum width (i.e. perpendicular to flow) allowable for the filter strip and design for that width or larger.

Step 2: Calculate the minimum width			
2-1. Enter the stormwater quality design flow (cfs), *SQDF*	SQDF =	1.18	cfs
2-2. Enter the minimum linear unit application rate (0.005 cfs/ft), $q_{a,min}$	$q_{a,min}$=	0.005	cfs/ft
2-3. Calculate the minimum width of filter strip (ft), $W_{min}=SQDF/q_{a,min}$	W_{min}=	240	ft

Step 3: Calculate the Design Flow Depth

A slope of 3% was assumed for the filter strip (2-4% recommended). The design water depth should not exceed 1 inch. For this design example a manning's coefficient of 0.27 was used.

Step 3: Calculate the design flow depth	
3-1. Enter filter strip longitudinal slope, *s* (ft/ft)	s = 0.03 ft/ft
3-2. Enter Manning roughness coefficient (0.25-0.30), n_{wq}	n_{wq} = 0.27
3-3. Enter width of strip (=impervious surface contributing area perpendicular to flow), at least W_{min} (ft), *W*	W = 240 ft
3-4. Calculate average depth of water using Manning equation (inches), $d_f = 12 * [SQDF * n_{wq} / 1.49 W s^{0.5}]^{0.6}$	d_f = 0.51 in
3-5. If d_f > 1" (0.083 ft), go back step 3-1 and decrease the slope	
3-6. If the slope cannot be changed due to construction constraints, go to step 3-3 and increase the width perpendicular to flow.	

Step 4: Calculate the Design Velocity

The designed flow velocity should not exceed 1 foot/second across the filter strip.

Step 4: Calculate the design velocity	
4-1. Enter depth of water (ft), $d_{f,ft} = d_f / 12$	d_f = 0.043 ft
4-2. Enter width of strip (ft), *W*	W= 240 ft
4-3. Calculate design flow velocity (ft/s), $V_{wq} = SQDF/(d_{f,ft} W)$	V_{wq} = 0.11 ft/s
4-4. If the V_{wq} >1 ft/s, go back to step 3-1 and decrease the slope.	

Step 5: Calculate the Length of the Filter Strip

The filter strip should be at least 4 feet long (in the direction of flow) and accommodate a minimum residence time of 7 minutes to provide adequate water quality treatment.

Step 5: Calculate the length of the filter strip	
5-1. Enter desired residence time (minimum 10 minutes), *t*	t = 10 min
5-2. Enter design flow velocity (ft/s), V_{wq}	V_{wq}= 0.11 ft/s
5-3. Calculate length of the filter strip (ft), $L = 60tV_{wq}$	L = 66 ft
5-4. If L < 4 ft, go to step 3-1 and increase the slope	

E.8 TCM-1 Dry Extended Detention Basin

Sizing Methodology

Dry extended detention (ED) basins are basins designed such that the stormwater quality design volume, SQDV, is detained for 36 to 48 hours. This allows sediment particles and associated pollutants to settle and be removed from stormwater. Procedures for sizing extended detention basins are summarized below. A sizing example is also provided.

Step 1: Calculate the design volume

Dry extended detention facilities shall be sized to capture and treat the water quality design volume (see Section E.1).

Step 2: Calculate the volume of the active basin

The total basin volume shall be increased an additional 20% of the stormwater quality design volume to account for sediment accumulation, at a minimum. If the basin is designed only for water quality treatment then the basin volume would be 120% of the stormwater quality design volume, SQDV. Freeboard is in additional to the total basin volume. Calculate the volume of the active basin, V_a:

$$V_a = 1.20 * SQDV \qquad \text{(Equation E-39)}$$

Step 3: Determine detention basin location and preliminary geometry based on site constraints

Based on site constraints, determine the basin geometry and the storage available by developing an elevation-storage relationship for the basin. The cross-sectional geometry across the width of the basin shall be approximately trapezoidal with a maximum side slope of 4:1 (H:V) on interior slopes and 3:1 (H:V) on exterior slopes unless specifically permitted by Ventura County (see Side Slopes below). Shallower side slopes are necessary if the basin is designed to have recreational uses during dry weather conditions.

1) Calculate the width of the basin footprint, W_{tot}, as follows:

$$W_{tot} = \frac{A_{tot}}{L_{tot}} \qquad \text{(Equation E-40)}$$

Where:

A_{tot} = total surface area of the basin footprint (ft^2)

L_{tot} = total length of the basin footprint (ft)

2) Calculate the length of the active volume surface area including the internal berm but excluding the freeboard, L_{av-tot}:

$$L_{av-tot} = L_{tot} - 2Zd_{fb} \quad \text{(Equation E-41)}$$

Where:

Z = interior side slope as length per unit height

d_{fb} = freeboard depth

3) Calculate the width of the active volume surface area including the internal berm but excluding freeboard, W_{av-tot}:

$$W_{av-tot} = W_{tot} - 2Zd_{fb} \quad \text{(Equation E-42)}$$

4) Calculate the total active volume surface area including the internal berm and excluding freeboard, A_{av-tot}:

$$A_{av-tot} = L_{av-tot} \times W_{av-tot} \quad \text{(Equation E-43)}$$

5) Calculate the area of the berm, A_{berm}:

$$A_{berm} = W_{berm} \times L_{berm} \quad \text{(Equation E-44)}$$

Where:

W_{berm} = width of the internal berm

L_{berm} = length of the internal berm

6) Calculate the surface area excluding the internal berm and freeboard, A_{av}:

$$A_{av} = A_{av=tot} - A_{berm} \quad \text{(Equation E-45)}$$

Step 4: Determine Dimensions of Forebay

5-15% of the basin active volume, V_a, is required to be within the active volume of the forebay.

1) Calculate the active volume of forebay, V_1:

$$V_1 = \frac{V_a \times \%V_1}{100} \quad \text{(Equation E-46)}$$

Where:

$\%V_1$ = percent of V_a in forebay (%)

V_a = active volume (ft^3)

2) Calculate the surface area for the active volume of forebay, A_1:

$$A_1 = \frac{V_1}{d_1} \qquad \text{(Equation E-47)}$$

Where:

d_1 = average depth for the active volume of forebay (ft)

3) Calculate the length of forebay, L_1:

$$L_1 = \frac{A_1}{W_1} \qquad \text{(Equation E-48)}$$

Where:

W_1 = width of forebay (ft)

Step 5: Determine Dimensions of Cell 2

Cell 2 will consist of the remainder of the basin's active volume.

1) Calculate the active volume of Cell 2, V_2:

$$V_2 = V_a - V_1 \qquad \text{(Equation E-49)}$$

Where:

V_a = total basin active volume (ft^3)

V_1 = volume of forebay (ft^3)

2) Calculate the surface area, A_2, for the active volume of Cell 2:

$$A_2 = A_{av} - A_1 \qquad \text{(Equation E-50)}$$

Where:

A_{av} = basin surface area excluding berm and freeboard (ft^2)

A_1 = surface area of forebay (ft^2)

3) Calculate the average depth, d_2, for the active volume of Cell 2:

$$d_2 = \frac{V_2}{A_2} \qquad \text{(Equation E-51)}$$

4) Calculate the length of Cell 2, L_2:

$$L_2 = \frac{A_2}{W_2} \qquad \text{(Equation E-52)}$$

Where:

W_2 = width of Cell 2 (ft)

5) Verify that the length-to-width ratio of Cell 2 at half of d_2 is at least 1.5:1 with ≥ 2:1 preferred. If the length-to width ratio is less than 1.5:1, modify input parameters until a ratio of at least 1.5:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the basin should be chosen. Calculate the length-to width, LW_{mid2}, ratio of Cell 2 at half of d_2 follows:

$$LW_{mid2} = \frac{L_{mid2}}{W_{mid2}} \qquad \text{(Equation E-53)}$$

Where:

$W_{mid2} = W_2 - Zd_2$ and (Equation E-54)

$L_{mid2} = L_2 - Zd_2$ (Equation E-55)

W_{mid2} = width of Cell 2 at half of d_2 (ft)

L_{mid2} = length of Cell 2 at half of d_2 (ft)

Z = interior side slope as length per unit height (H:V)

Step 6: Ensure Design Requirements and Site Constraints are achieved

Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location or alternative treatment BMP.

Step 7: Size Outlet Structure

The total drawdown time for the basin should be 36-48 hours. The outlet structure shall be designed to release the bottom 50% of the detention volume (half-full to empty) over 24-32 hours, and the top half (full to half-full) in 12-16 hours. A primary overflow should be sized to pass the peak flow rate from the developed capital design storm. See Section 6 for outlet structure sizing methodologies.

Step 8: Determine Emergency Spillway Requirements

For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm in order to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the 100-yr, 24-hr post-development peak storm water runoff discharge rate directly to the downstream conveyance system or another acceptable discharge point. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

Sizing Worksheet

Step 1: Determine water quality design volume		
1-1. Enter Project area (acres), $A_{project}$	A =	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	%
1-3. Determine the maximum allowed effective impervious area (ac), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA=	acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p (1-imp)$	C =	
1-9. Enter design rainfall depth of the storm (in), P_i	P_i =	in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P =	ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560 \bullet C*P*A_{retain}$	SQDV =	ft^3
Step 2: Calculate the volume of the active basin		
2-1. Calculate basin active volume (includes water quality design volume + sediment storage volume) (ft^3), $V_a = 1.20 \times SQDV$	V_a =	ft^3

Step 3: Determine Detention Basin Location and Preliminary Geometry Based on Site Constraints		
3-1. Based on site constraints, determine the basin geometry and the storage available by developing an elevation-storage relationship for the basin. For this simple example, assume a trapezoidal geometry for cell 1 (forebay) and cell 2.		
3-2. Enter the total surface area of the basin footprint based on site constraints (ft²), A_{tot}	A_{tot} =	ft²
3-3. Enter the length of the basin footprint based on site constraints (ft), L_{tot}	L_{tot} =	ft
3-4. Calculate the width of the basin footprint (L:W = 1.5:1 min) (ft), $W_{tot} = A_{tot} / L_{tot}$	W_{tot} =	ft
3-5. Enter interior side slope as length per unit height (H:V, min = 3), Z	Z =	
3-6. Enter desired freeboard depth (ft), d_{fb} (min: 2 ft on-line; 1 ft offline)	d_{fb} =	ft
3-7. Calculate the length of the active volume surface area including the internal berm but excluding freeboard, $L_{av\text{-}tot} = L_{tot} - 2Zd_{fb}$	$L_{av\text{-}tot}$ =	ft
3-8. Calculate the width of the active volume surface area including the internal berm but excluding freeboard, $W_{av\text{-}tot} = W_{tot} - 2Zd_{fb}$	$W_{av\text{-}tot}$ =	ft
3-9. Calculate the total active volume surface area including the internal berm and excluding freeboard, $A_{av\text{-}tot} = L_{av\text{-}tot} \times W_{av\text{-}tot}$	$A_{av\text{-}tot}$ =	ft²
3-10. Enter the width of the internal berm (6 ft min), W_{berm}	W_{berm} =	ft
3-11. Enter the length of the internal berm (ft), $L_{berm} = W_{av\text{-}tot}$	L_{berm} =	ft
3-12. Calculate the area of the berm (ft²), $A_{berm} = W_{berm} \times L_{berm}$	A_{berm} =	ft²
3-13. Calculate the surface area excluding the internal berm and freeboard (ft²), $A_{av} = A_{av\text{-}tot} - A_{berm}$	A_{av} =	ft²

Step 4: Determine Dimensions of forebay		
4-1. Enter the percent of V_a in forebay (5-15% required), $\%V_1$	$\%V_1$ =	%
4-2. Calculate the active volume of forebay, $V_1 = (V_a \bullet \%V_1)/100$	V_1 =	ft^3
4-3. Enter a desired average depth for the active volume of forebay, d_1	d_1 =	ft
4-4. Calculate the surface area for the active volume of forebay, $A_1 = V_1 / d_1$	A_1 =	ft^2
4-5. Enter the width of forebay, $W_1 = W_{av\text{-}tot} = L_{berm}$	W_1 =	ft
4-6. Calculate the length of forebay (<u>Note:</u> inlet and outlet should be configured to maximize the residence time), $L_1 = A_1 / W1$	L_1 =	ft
Step 5: Determine Dimensions of Cell 2		
5-1. Calculate the active volume of Cell 2, $V_2 = V_a - V_1$	V_2 =	ft^3
5-2. Calculate the surface area of the active volume of Cell 2, $A_2 = A_{av} - A_1$	A_2 =	ft^2
5-3. Calculate the average depth for the active volume of Cell 2, $d_2 = V_2 / A_2$	d_2 =	ft
5-4. Enter the width of Cell 2, $W_2 = W_1 = W_{av\text{-}tot} = L_{berm}$	W_2 =	ft
5-5. Calculate the length of Cell 2, $L_2 = A_2 / W_2$	L_2 =	ft
5-6. Calculate the width of Cell 2 at half of d_2, $W_{mid2} = W_2 - Zd_2$	W_{mid2} =	ft
5-7. Calculate the length of Cell 2 at half of $d_{2,}$ $L_{mid2} = L_2 - Zd_2$	L_{mid2} =	ft

5-8. Verify that the length-to-width ratio of Cell 2 at half of d_2 is at least 1.5:1 with ≥ 2:1 preferred. If the length-to-width ratio is less than 1.5:1, modify input parameters until a ratio of at least 1.5:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the basin should be chosen, $LW_{mid2} = L_{mid2} / W_{mid2}$	LW_{mid2} =
Step 6: Ensure Design Requirements and Site Constraints are Achieved	
6-1. Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location or alternative treatment BMP.	
Step 7: Size Outlet Structure	
7-1. The total drawdown time for the basin should be 36-48 hours. The outlet structure shall be designed to release the bottom 50% of the detention volume (half-full to empty) over 24-32 hours, and the top half (full to half-full) in 12-16 hours. A primary overflow should be sized to pass the peak flow rate from the developed capital design storm. See Section 6 for outlet structure sizing methodologies.	
Step 8: Determine Emergency Spillway Requirements	
8-1. For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm in order to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the 100-yr, 24-hr post-development peak storm water runoff discharge rate directly to the downstream conveyance system or another acceptable discharge point. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.	

Design Example

Step 1: Determine water quality design volume

For this design example, a 10-acre residential development with a 60% total impervious area is considered. The 85th percentile storm event for the project location is 0.75 inches.

Step 1: Determine water quality design volume			
1-1. Enter Project area (acres), $A_{project}$	A =	10	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	5	
1-3. Determine the maximum allowed effective impervious area (ac), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	0.5	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	0.6	
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA=	6	acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	5.5	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	0.05	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\,(1-imp)$	C =	0.59	
1-9. Enter design rainfall depth of the storm (in), P_i	P_i =	0.75	in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P =	0.06	ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560 \bullet C*P*A_{retain}$	SQDV =	8,500	ft^3

Step 2: Calculate Volume of the Active Basin and the Forebay Basin

Step 2: Calculate the design volume of the active basin	
2-1. Calculate basin active design volume (includes water quality design volume + sediment storage volume), $V_a = 1.20*SQDV$	V_a = 10,000 ft^3

Step 3: Determine Detention Basin Location and Preliminary Geometry Based on Site Constraints

The detention basin in this example has an internal berm separating the forebay (Cell 1) and the main basin (Cell 2). The internal berm elevation is 2 ft below the elevation of the SUSMP volume within the entire basin. The berm length is equal to the width of the basin when filled to the active design volume.

Step 3: Determine Detention Basin Location and Preliminary Geometry Based on Site Constraints	
3-1. Based on site constraints, determine the basin geometry and the storage available by developing an elevation-storage relationship for the basin. For this simple example, assume a trapezoidal geometry for cell 1 (forebay) and cell 2.	
3-2. Enter the total surface area of the basin footprint based on site constraints, A_{tot}	A_{tot} = 8,000 ft^2
3-3. Enter the length of the basin footprint based on site constraints, L_{tot} (L:W = 1.5:1 min)	L_{tot} = 200 ft
3-4. Calculate the width of the basin footprint, $W_{tot} = A_{tot} / L_{tot}$	W_{tot} = 40 ft
3-5. Enter interior side slope as length per unit height (min = 3), Z	Z = 3
3-6. Enter desired freeboard depth, d_{fb} (min: 2 ft on-line; 1 ft offline)	d_{fb} = 2 ft
3-7. Calculate the length of the active volume surface area including the internal berm but excluding freeboard, $L_{av\text{-}tot} = L_{tot} - 2Zd_{fb}$	$L_{av\text{-}tot}$ = 188 ft

Step 3: Determine Detention Basin Location and Preliminary Geometry Based on Site Constraints	
3-8. Calculate the width of the active volume surface area including the internal berm but excluding freeboard, $W_{av\text{-}tot} = W_{tot} - 2Zd_{fb}$	$W_{av\text{-}tot}$ = 28 ft
3-9. Calculate the total active volume surface area including the internal berm and excluding freeboard, $A_{av\text{-}tot} = L_{av\text{-}tot} \bullet W_{av\text{-}tot}$	$A_{av\text{-}tot}$ = 5,300 ft^2
3-10. Enter the width of the internal berm (6 ft min), W_{berm}	W_{berm} = 6 ft
3-11. Enter the length of the internal berm, $L_{berm} = W_{av\text{-}tot}$	L_{berm} = 28 ft
3-12. Calculate the area of the berm, $A_{berm} = W_{berm} \bullet L_{berm}$	A_{berm} = 170 ft^2
3-13. Calculate the surface area excluding the internal berm and freeboard, $A_{av} = A_{av\text{-}tot} - A_{berm}$	A_{av} = 5,130 ft^2

Step 4: Calculate Dimensions of Cell 1

Calculate the dimensions of the forebay (Cell 1) based on the active design volume for Cell 1 (25% of V_a) and a desired average depth, d_1. The width of the forebay, W_1, is equivalent to the length of the berm, L_{berm}, and the width of Cell 2, W_2.

Step 4: Determine Dimensions of forebay	
4-1. Enter the percent of V_a in forebay (5-15% required), $\%V_1$	$\%V_1$ = 25 %
4-2. Calculate the active volume of forebay (including sediment storage), $V_1 = (V_a \bullet \%V_1)/100$	V_1 = 2,500 ft^3
4-3. Enter a desired average depth for the active volume of forebay, d_1	d_1 = 5 ft
4-4. Calculate the surface area for the active volume of forebay, $A_1 = V_1 / d_1$	A_1 = 500 ft^2

4-5. Enter the width of forebay, $W_1 = W_{wq\text{-}tot} = L_{berm}$	W_1 = 28 ft
4-6. Calculate the length of forebay (<u>Note:</u> inlet and outlet should be configured to maximize the residence time), $L_1 = A_1 / W1$	L_1 = 18 ft

Step 5: Calculate the Dimensions of Cell 2

Calculate the dimensions of the main basin (Cell 2) based on the active design volume for Cell 2 and a desired average depth, d_2. A calculation of the length, L_{mid2}, and width, W_{mid2}, at half basin depth, d_2, is conducted in order to verify that the length-to-width ratio at half d_2 is greater than 1.5:1.

Step 5: Calculate the dimensions of Cell 2	
5-1. Calculate the active volume of Cell 2, $V_2 = V_a - V_1$	V_2 = 7,500 ft^3
5-2. Calculate the surface area of the active volume of Cell 2, $A_2 = A_{av} - A_1$	A_2 = 4,630 ft^2
5-3. Calculate the average depth of the active volume of Cell 2, $d_2 = V_2 / A_2$	d_2 = 1.6 ft
5-4. Enter the width of Cell 2, $W_2 = W_1 = W_{av\text{-}tot} = L_{berm}$	W_2 = 28 ft
5-5. Calculate the length of Cell 2, $L_2 = A_2 / W_2$	L_2 = 166 ft
5-6. Calculate the width of Cell 2 at half of d_2, $W_{mid2} = W_2 - Zd_2$	W_{mid2} = 23 ft
5-7. Calculate the length of Cell 2 at half of d_2, $L_{mid2} = L_2 - Zd_2$	L_{mid2} = 161 ft
5-8. Verify that the length-to-width ratio of Cell 2 at half of d_2 is at least 1.5:1 with ≥ 2:1 preferred. If the length-to-width ratio is less than 1.5:1, modify input parameters until a ratio of at least 1.5:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the basin should be chosen, $LW_{mid2} = L_{mid2} / W_{mid2}$	LW_{mid2} = 7

Step 6: Ensure Design Requirements and Site Constraints are Achieved

Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location or an alternative treatment BMP.

Step 7: Size Outlet Structure

The total drawdown time for the basin should be 36-48 hours. The outlet structure shall be designed to release the bottom 50% of the detention volume (half-full to empty) over 24-32 hours, and the top half (full to half-full) in 12-16 hours. A primary overflow should be sized to pass the peak flow rate from the developed capital design storm. See Section 6 for outlet structure sizing methodologies.

Step 8: Determine Emergency Spillway Requirements

For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm in order to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the 100-yr, 24-hr post-development peak storm water runoff discharge rate directly to the downstream conveyance system or another acceptable discharge point. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

E.9 TCM-2 Wet Detention Basin

Sizing Methodology

Wet Detention basins may be designed with or without extended detention above the permanent pool. The extended detention portion of the wet detention basin above the permanent pool, if provided, functions like a dry extended detention (ED) basin (see VEG-5: Dry Extended Detention Basin). If there is no extended detention provided, wet detention basins shall be sized to provide a minimum wet pool volume equal to the stormwater quality design volume plus an additional 5% for sediment accumulation. If extended detention is provided above the permanent pool, the sizing is dependent of the functionality of the basin; the basin may function as water quality treatment only or water quality plus peak flow attenuation.

If and the basin is designed for water quality treatment only, then the permanent pool volume shall be a minimum of 10 percent of the stormwater quality design volume and the surcharge volume (above the permanent pool) shall make up the remaining 90 percent. If extended detention is provided above the permanent pool and the basin is designed for water quality treatment and peak flow attenuation, then the permanent pool volume shall be equal to the water quality treatment volume, and the surcharge volume shall be sized to attenuate peak flows in order to meet the peak runoff discharge requirements. The extended detention portion of the wet detention basin above the permanent pool, if provided, functions like a dry extended detention (ED) basin (see VEG-5: Dry Extended Detention Basin).

Step 1: Calculate the design volume

Wet detention basins shall be sized with a permanent pool volume equal to the SQDV volume (see Section 2 and Appendix E).

Step 2: Determine the active design volume for the wet detention basin without extended detention

The active volume of the wet detention basin, V_a, shall be equal to the SQFV plus an additional 5% for sediment accumulation.

$$V_a = 1.05 \times SQDV \qquad \text{(Equation E-56)}$$

Step 3: Determine pond location and preliminary geometry based on site constraints

Based on site constraints, determine the pond geometry and the storage available by developing an elevation-storage relationship for the pond. Note that a more natural geometry may be used and is in many cases recommended; the preliminary basin geometry calculations should be used for sizing purposes only.

1) Calculate the width of the pond footprint, W_{tot}, as follows:

$$W_{tot} = \frac{A_{tot}}{L_{tot}} \quad \text{(Equation E-57)}$$

Where:

A_{tot} = total surface area of the pond footprint (ft^2)

L_{tot} = total length of the pond footprint (ft)

7) Calculate the length of the active volume surface area including the internal berm but excluding the freeboard, $L_{av\text{-}tot}$:

$$L_{av-tot} = L_{tot} - 2Zd_{fb} \quad \text{(Equation E-58)}$$

Where:

Z = interior side slope as length per unit height

d_{fb} = freeboard depth

8) Calculate the width of the active volume surface area including the internal berm but excluding freeboard, $W_{av\text{-}tot}$:

$$W_{av-tot} = W_{tot} - 2Zd_{fb} \quad \text{(Equation E-59)}$$

9) Calculate the total active volume surface area including the internal berm and excluding freeboard, $A_{av\text{-}tot}$:

$$A_{av-tot} = L_{av-tot} \times W_{av-tot} \quad \text{(Equation E-60)}$$

10) Calculate the area of the berm, A_{berm}:

$$A_{berm} = W_{berm} \times L_{berm} \quad \text{(Equation E-61)}$$

Where:

W_{berm} = width of the internal berm

L_{berm} = length of the internal berm

11) Calculate the active volume surface area excluding the internal berm and freeboard, A_{wq}:

$$A_{wq} = A_{wq=tot} - A_{berm} \quad \text{(Equation E-62)}$$

Step 4: Determine Dimensions of Forebay

The wet detention basin shall be divided into two cells separated by a berm or baffle. The forebay shall contain between 5 and 10 percent of the total volume. The berm or baffle volume shall not count as part of the total volume. Calculate the active volume of forebay, V_1:

$$V_1 = \frac{V_a \times \%V_1}{100} \qquad \text{(Equation E-63)}$$

Where:

$\%V_1$ = percent of SQDV in forebay (%)

1) Calculate the surface area for the active volume of forebay, A_1:

$$A_1 = \frac{V_1}{d_1} \qquad \text{(Equation E-64)}$$

Where:

d_1 = average depth fo rhte active volume of forebay (ft)

2) Calculate the length of forebay, L_1. Note, inlet and outlet should be configured to maximize the residence time.

$$L_1 = \frac{A_1}{W_1} \qquad \text{(Equation E-65)}$$

Where:

W_1 = width of forebay (ft), $W_1 = W_{av\text{-}tot} = L_{berm}$

Step 5: Determine Dimensions of Cell 2

Cell 2 will consist of the remainder of the basin's active volume.

3) Calculate the active volume of Cell 2, V_2:

$$V_2 = V_a - V_1 \qquad \text{(Equation E-66)}$$

4) The minimum wetpool surface area includes 0.3 acres of wetpool per acre-foot of permanent wetpool volume. Calculate A_{min2}:

$$A_{min2} = (V_2 \times 0.3 \frac{acres}{acre-feet}) \qquad \text{(Equation E-67)}$$

5) Calculate the actual wetpool surface area, A_2:

$$A_2 = A_{av} - A_1 \qquad \text{(Equation E-68)}$$

Verify that A_2 is greater than A_{min2}. If A_2 is less than A_{min2}, then modify input parameters to increase A_2 until it is greater than A_{min2}. If site constraints limit this criterion, then another site for the pond should be chosen.

6) Calculate the top length of Cell 2, L_2:

$$L_2 = \frac{A_2}{W_2} \quad \text{(Equation E-69)}$$

Where:

W_2 = width of Cell 2 (ft), $W_2 = W_1 = W_{wq\text{-}tot} = L_{berm}$

7) Verify that the length-to-width ratio of Cell 2 is at least 1.5:1 with ≥ 2:1 preferred. If the length-to-width ratio is less than 1.5:1, modify input parameters until a ratio of at least 1.5:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the pond should be chosen.

$$LW_2 = \frac{L_2}{W_2} \quad \text{(Equation E-70)}$$

8) Calculate the emergent vegetation surface area, A_{ev}:

$$A_{ev} = \frac{A_2 \bullet \%A_{ev}}{100} \quad \text{(Equation E-71)}$$

Where:

$\%A_{ev}$ = percent of surface area that will be planted with emergent vegetation

9) Calculate the volume of the emergent vegetation shallow zone (1.5 – 3 ft), V_{ev}:

$$V_{ev} = A_{ev} \bullet d_{ev} \quad \text{(Equation E-72)}$$

Where:

d_{ev} = average depth of the emergent vegetation shallow zone (1.5 – 3 ft)

10) Calculate the length of the emergent vegetation shallow zone, L_{ev}:

$$L_{ev} = \frac{A_{ev}}{W_{ev}} \quad \text{(Equation E-73)}$$

Where:

W_{ev} = width of the emergent vegetation shallow zone (ft), $W_{ev} = W_2$

11) Calculate the volume of the deep zone, V_{deep}:

$$V_{deep} = V_2 - V_{ev} \quad \text{(Equation E-74)}$$

12) Calculate the surface area of the deep (>3 ft) zone, A_{deep}:

$$A_{deep} = A_2 - A_{ev} \qquad \text{(Equation E-75)}$$

13) Calculate the average depth of the deep zone (4-8 ft), d_{deep}:

$$d_{deep} = \frac{V_{deep}}{A_{deep}} \qquad \text{(Equation E-76)}$$

14) Calculate length of the deep zone, L_{deep}:

$$L_{deep} = \frac{A_{deep}}{W_{deep}} \qquad \text{(Equation E-77)}$$

Where:

W_{deep} = width of the deep zone (ft), $W_{deep} = W_2$

Step 6: Ensure design requirements and site constraints are achieved

Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location for the BMP.

Step 7: Size Outlet Structure

For extended detention wet detention basin, outlet structures shall be designed to provide 12 to 48 hour emptying time for the water quality volume above the permanent pool.

The basin outlet pipe shall be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for off-line basins or flows greater than the peak runoff discharge rate for the 100-year, 24-hr design storm for on-line basins.

Step 8: Determine Emergency Spillway Requirements

For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the water quality design storm. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

Sizing Worksheet

Step 1: Determine water quality design volume		
1-1. Enter drainage area, A	A =	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	%
1-3. Determine the maximum allowed effective impervious area, $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	
1-5. Determine the Project Total Impervious area, $TIA=A_{project}*Imp$	TIA=	acres
1-6. Determine the total area from which runoff must be retained, $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p (1-imp)$	C =	
1-9. Enter design rainfall depth of the storm, P_i (in)	P_i =	in
1-10. Calculate rainfall depth, $P = P_i/12$	P =	ft
1-11. Calculate water quality design volume, $SQDV = 43560 \bullet P*A_{retain}*C$	SQDV =	ft^3
Step 2: Determine active design volume for the wet pond without extended detention		
2-1. Calculate the active design volume (without extended detention), $V_a = 1.05*SQDV$	V_a =	ft^3

Step 3: Determine Pond Location and Preliminary Geometry Based on Site Constraints	
3-1. Based on site constraints, determine the pond geometry and the storage available by developing an elevation-storage relationship for the pond. For this simple example, assume a trapezoidal geometry for cell 1 (forebay) and cell 2.	
3-2. Enter the total surface area of the pond footprint based on site constraints, A_{tot}	A_{tot} = ft^2
3-3. Enter the length of the pond footprint based on site constraints, L_{tot}	L_{tot} = ft
3-4. Calculate the width of the pond footprint, $W_{tot} = A_{tot} / L_{tot}$	W_{tot} = ft
3-5. Enter interior side slope as length per unit height (min = 3), Z	Z =
3-6. Enter desired freeboard depth, d_{fb} (1 ft min)	d_{fb} = ft
3-7. Calculate the length of the water quality volume surface area including the internal berm but excluding freeboard, $L_{av\text{-}tot} = L_{tot} - 2Zd_{fb}$	$L_{av\text{-}tot}$ = ft
3-8. Calculate the width of the water quality volume surface area including the internal berm but excluding freeboard, $W_{av\text{-}tot} = W_{tot} - 2Zd_{fb}$	$W_{av\text{-}tot}$ = ft
3-9. Calculate the total water quality volume surface area including the internal berm and excluding freeboard, $A_{av\text{-}tot} = L_{av\text{-}tot} \cdot W_{av\text{-}tot}$	$A_{av\text{-}tot}$ = ft^2
3-10. Enter the width of the internal berm (6 ft min), W_{berm}	W_{berm} = ft
3-11. Enter the length of the internal berm, $L_{berm} = W_{av\text{-}tot}$	L_{berm} = ft
3-12. Calculate the area of the berm, $A_{berm} = W_{berm} \cdot L_{berm}$	A_{berm} = ft^2

3-13. Calculate the water quality volume surface area excluding the internal berm and freeboard, $A_{av} = A_{av\text{-}tot} - A_{berm}$	A_{av} = ft^2
Step 4: Determine Dimensions of forebay	
4-1. Enter the percent of V_a in forebay (5-10% required), $\%V_1$	$\%V_1$ = %
4-2. Calculate the active volume of forebay (includes sediment storage volume), $V_1 = (V_a \bullet \%V_1) / 100$	V_1 = ft^3
4-3. Enter desired average depth of forebay (5-9 ft including sediment storage of 1 ft), d_1	d_1 = ft
4-4. Calculate the surface area for the active volume of forebay, $A_1 = V_1 / d_1$	A_1 = ft^2
4-5. Enter the width of forebay, $W_1 = W_{av\text{-}tot} = L_{berm}$	W_1 = ft
4-6. Calculate the length of forebay (<u>Note:</u> inlet and outlet should be configured to maximize the residence time), $L_1 = A_1 / W1$	L_1 = ft
Step 5: Determine Dimensions of Cell 2	
5-1. Calculate the active volume of Cell 2, $V_2 = V_a - V_1$	V_2 = ft^3
5-2. Determine minimum wetpool surface area, $A_{min2} = V_2 \bullet 0.3$	A_{min2} = ft^2
5-3. Determine actual wetpool surface area, $A_2 = A_{av} - A_1$	A_2 = ft^2
5-4. • If A_2 is greater than A_{min2} then move on to step 5-5. • If A_2 is less than A_{min2}, then modify input parameters to increase A_2 until it is greater than A_{min2}. If site constraints limit this criterion, then another site for the pond should be chosen.	
5-5. Enter width of Cell 2, $W_2 = W_1 = W_{av\text{-}tot} = L_{berm}$	W_2 = ft

5-6. Calculate top length of Cell 2, $L_2 = A_2 / W_2$	L_2 = ft
5-7. Verify that the length-to-width ratio of Cell 2 is at least 1.5:1 with ≥ 2:1 preferred. If the length-to-width ratio is less than 1.5:1, modify input parameters until a ratio of at least 1.5:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the pond should be chosen, $LW_2 = L_2 / W_2$	LW_2 =
5-8. Enter percent of surface area that will be planted with emergent vegetation (25-75%), $\%A_{ev}$	$\%A_{ev}$ = %
5-9. Calculate emergent vegetation surface area, $A_{ev} = (A_2 \bullet \%A_{ev})/100$	A_{ev} = ft^2
5-10. Enter average depth of emergent vegetation shallow zone (1.5 – 3 ft), d_{ev}	d_{ev} = ft
5-11. Calculate volume of emergent vegetation shallow zone (1.5 – 3 ft), $V_{ev} = A_{ev} \bullet d_{ev}$	V_{ev} = ft^3
5-12. Enter width of emergent vegetation shallow zone, $W_{ev} = W_2$	W_{ev}= ft
5-13. Calculate length of emergent vegetation shallow zone, $L_{ev} = A_{ev} / W_{ev}$	L_{ev} = ft
5-14. Calculate volume of deep zone, $V_{deep} = V_2 – V_{ev}$	V_{deep} = ft^3
5-15. Calculate surface area of deep (>3 ft) zone, $A_{deep} = A_2 – A_{ev}$	A_{deep} = ft^2
5-16. Calculate average depth of deep zone (4 - 8 ft), $d_{deep} = V_{deep} / A_{deep}$	d_{deep} = ft
5-17. Enter width of deep zone, $W_{deep} = W_2$	W_{deep} = ft
5-18. Calculate length of deep zone, $L_{deep} = A_{deep} / W_{deeo}$	L_{deep} = ft

Step 6: Ensure Design Requirements and Site Constraints are Achieved
6-1. Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location for the BMP.
Step 7: Size Outlet Structure
7-1. The basin outlet pipe shall be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for off-line basins or flows greater than the peak runoff discharge rate for the 100-year, 24-hr design storm for on-line basins.
Step 8: Determine Emergency Spillway Requirements
8-1. For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the water quality design storm. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

Design Example

Wet detention basin siting requires the following considerations prior to construction: (1) availability of base flow – wet detention basins require a regular source of water if water level is to be maintained, (2) surface space availability – large footprint area is required, and (3) compatibility with flood control – basins must not interfere with flood control functions of existing conveyance and detention structures.

The wet detention basin in this example does not have extended detention. An internal berm separates the forebay (Cell 1) and the main basin (Cell 2). The berm is at the elevation of the active volume design surface which is also the permanent wetpool elevation.

Step 1: Determine Water Quality Design Volume

For this design example, a 20-acre residential development with a 60% total impervious area is considered. The 85th percentile storm event for the project location is 0.75 inches.

Step 1: Determine water quality design volume			
1-1. Enter drainage area, A	A =	20	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	5	
1-3. Determine the maximum allowed effective impervious area, $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	1.0	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	0.6	
1-5. Determine the Project Total Impervious area, $TIA=A_{project}*Imp$	TIA=	12	acres
1-6. Determine the total area from which runoff must be retained, $A_{retain}=TIA\text{-}EIA_{allowable}$	A_{retain} =	11	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	0.05	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\ (1\text{-}imp)$	C =	0.59	
1-9. Enter design rainfall depth of the storm, P_i (in)	P_i =	0.75	in
1-10. Calculate rainfall depth, $P = P_i/12$	P =	0.06	ft

1-11. Calculate water quality design volume, $SQDV = 43560 \cdot P * A_{retain} * C$	SQDV = 17,000 ft^3

Step 2: Determine Active Design Volume for a Wet Detention Basin without Extended Detention

If there is no extended detention provided, wet detention basins shall be sized to provide a minimum wet pool volume equal to the water quality design volume plus an additional 5% for sediment accumulation.

Step 2: Determine Active Design Volume for a Wet Detention Basin without Extended Detention	
2-1. Calculate the active design volume (without extended detention), $V_a = 1.05 * SQDV$	V_a = 17,800 ft^3

Step 3: Determine Pond Location and Preliminary Geometry Based on Site Constraints

A total footprint area and total length available for the basin is provided. This step calculates the total active volume surface area which is equivalent to the permanent wetpool surface area. This step also calculates the dimensions of the internal berm.

Step 3: Determine Pond Location and Preliminary Geometry Based on Site Constraints	
3-1. Based on site constraints, determine the pond geometry and the storage available by developing an elevation-storage relationship for the pond. For this simple example, assume a trapezoidal geometry for cell 1 (forebay) and cell 2.	
3-2. Enter the total surface area of the pond footprint based on site constraints, A_{tot}	A_{tot} = 7,500 ft^2
3-3. Enter the length of the pond footprint based on site constraints, L_{tot}	L_{tot} = 150 ft
3-4. Calculate the width of the pond footprint, $W_{tot} = A_{tot} / L_{tot}$	W_{tot} = 50 ft
3-5. Enter interior side slope as length per unit height (min = 3), Z	Z = 3

Step 3: Determine Pond Location and Preliminary Geometry Based on Site Constraints			
3-6. Enter desired freeboard depth, d_{fb} (1 ft min)	d_{fb} =	2	ft
3-7. Calculate the length of the water quality volume surface area including the internal berm but excluding freeboard, $L_{av\text{-}tot} = L_{tot} - 2Zd_{fb}$	$L_{av\text{-}tot}$ =	138	ft
3-8. Calculate the width of the water quality volume surface area including the internal berm but excluding freeboard, $W_{av\text{-}tot} = W_{tot} - 2Zd_{fb}$	$W_{av\text{-}tot}$ =	38	ft
3-9. Calculate the total water quality volume surface area including the internal berm and excluding freeboard, $A_{av\text{-}tot} = L_{av\text{-}tot} \bullet W_{av\text{-}tot}$	$A_{av\text{-}tot}$ =	4,940	ft^2
3-10. Enter the width of the internal berm (6 ft min), W_{berm}	W_{berm} =	6	ft
3-11. Enter the length of the internal berm, $L_{berm} = W_{av\text{-}tot}$	L_{berm} =	38	ft
3-12. Calculate the area of the berm, $A_{berm} = W_{berm} \bullet L_{berm}$	A_{berm} =	230	ft^2
3-13. Calculate the water quality volume surface area excluding the internal berm and freeboard, $A_{av} = A_{av\text{-}tot} - A_{berm}$	A_{av} =	4,710	ft^2

Step 4: Determine Dimensions of forebay

It should be assumed that the forebay should be 5-10% of the total active design volume, V_a.

Step 4: Determine Dimensions of Cell 1			
4-1. Enter the percent of V_a in forebay (5-10% required), $\%V_1$	$\%V_1$ =	20	%
4-2. Calculate the active volume of forebay (includes sediment storage volume), $V_1 = (V_a \bullet \%V_1) / 100$	V_1 =	3,560	ft^3
4-3. Enter desired average depth of forebay (5-9 ft including sediment storage of 1 ft), d_1	d_1 =	8	ft

4-4. Calculate the surface area for the active volume of forebay, $A_1 = V_1 / d_1$	A_1 = 440 ft²
4-5. Enter the width of forebay, $W_1 = W_{av\text{-}tot} = L_{berm}$	W_1 = 38 ft
4-6. Calculate the length of forebay (Note: inlet and outlet should be configured to maximize the residence time), $L_1 = A_1 / W1$	L_1 = 12 ft

Step 5: Determine Dimensions of Cell 2

Verify that the surface area and length-to-width ratio of Cell 2 meet the design criteria. Calculate volumes, depths and surface areas for the emergent vegetation shallow zone and the deep zone.

Step 5: Determine Dimensions of Cell 2	
5-1. Calculate the active volume of Cell 2, $V_2 = V_a - V_1$	V_2 = 14,200 ft³
5-2. Determine minimum wetpool surface area, $A_{min2} = V_2 \bullet 0.3$	A_{min2} = 4,270 ft²
5-3. Determine actual wetpool surface area, $A_2 = A_{av} - A_1$	A_2 = 4,270 ft²
5-4. If A_2 is greater than A_{min2} then move on to step 5-5. If A_2 is less than A_{min2}, then modify input parameters to increase A_2 until it is greater than A_{min2}. If site constraints limit this criterion, then another site for the pond should be chosen.	
5-5. Enter width of Cell 2, $W_2 = W_1 = W_{av\text{-}tot} = L_{berm}$	W_2 = 38 ft
5-6. Calculate top length of Cell 2, $L_2 = A_2 / W_2$	L_2 = 110 ft
5-7. Verify that the length-to-width ratio of Cell 2 is at least 1.5:1 with ≥ 2:1 preferred. If the length-to-width ratio is less than 1.5:1, modify input parameters until a ratio of at least 1.5:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the pond should be chosen, $LW_2 = L_2 / W_2$	LW_2 = 2.9
5-8. Enter percent of surface area that will be planted with emergent vegetation (25-75%), $\%A_{ev}$	$\%A_{ev}$ = 25 %

Step 5: Determine Dimensions of Cell 2			
5-9. Calculate emergent vegetation surface area, $A_{ev} = (A_2 \cdot \%A_{ev})/100$	A_{ev} =	1,070	ft^2
5-10. Enter average depth of emergent vegetation shallow zone (1.5 – 3 ft), d_{ev}	d_{ev} =	2	ft
5-11. Calculate volume of emergent vegetation shallow zone (1.5 – 3 ft), $V_{ev} = A_{ev} \cdot d_{ev}$	V_{ev} =	2,130	ft^3
5-12. Enter width of emergent vegetation shallow zone, $W_{ev} = W_2$	W_{ev}=	38	ft
5-13. Calculate length of emergent vegetation shallow zone, $L_{ev} = A_{ev} / W_{ev}$	L_{ev} =	56	ft
5-14. Calculate volume of deep zone, $V_{deep} = V_2 - V_{ev}$	V_{deep} =	13,100	ft^3
5-15. Calculate surface area of deep (>3 ft) zone, $A_{deep} = A_2 - A_{ev}$	A_{deep} =	3,200	ft^2
5-16. Calculate average depth of deep zone (4 - 8 ft), $d_{deep} = V_{deep} / A_{deep}$	d_{deep} =	4.1	ft
5-17. Enter width of deep zone, $W_{deep} = W_2$	W_{deep} =	28	ft
5-18. Calculate length of deep zone, $L_{deep} = A_{deep} / W_{deeo}$	L_{deep} =	114	ft

Step 6: Ensure Design Requirements and Site Conditions are Achieved

Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location for the BMP.

Step 7: Size Outlet Structure

The basin outlet pipe shall be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for off-line basins or flows greater than the peak runoff discharge rate for the 100-year, 24-hr design storm for on-line basins.

Step 8: Determine Emergency Spillway Requirements

For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the water quality design storm. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

E.10 TCM-3 Constructed Wetland

Sizing Methodology

In most cases, the constructed treatment wetland permanent pool shall be sized to be greater than or equal to the stormwater quality design volume. If extended detention is provided above the permanent pool and the wetland is designed for water quality treatment only, then the permanent pool volume shall be a minimum of 80 percent of the stormwater quality design volume and the surcharge volume (above the permanent pool) shall make up the remaining 20 percent and provide at least 12 hours of detention. If extended detention is provided and the basin is designed for water quality treatment and peak flow attenuation, then the permanent pool volume shall be equal to the water quality treatment volume and the surcharge volume shall be sized to attenuate peak flows to meet the peak runoff discharge requirements. The extended detention portion of the wetland above the permanent pool, if provided, functions like a dry extended detention (ED) basin (see VEG-5: Dry Extended Detention Basin).

Step 1: Calculate the design volume

Constructed wetlands shall be sized to be greater than or equal to the SQDV volume (see Section 2 and Appendix E).

Step 2: Determine the Wetland Location, Wetland Type and Preliminary Geometry Based on Site Constraints

Based on site constraints, determine the wetland geometry and the storage available by developing an elevation-storage relationship for the wetland. The equations provided below assume a trapezoidal geometry for cell 1 (Forebay) and cell 2, and assumes that the wetland does not have extended detention.

1) Calculate the width of the wetland footprint, W_{tot}, as follows:

$$W_{tot} = \frac{A_{tot}}{L_{tot}} \quad \text{(Equation E-78)}$$

Where:

A_{tot} = total surface area of the wetland footprint (ft^2)

L_{tot} = total length of the wetland footprint (ft)

12) Calculate the length of the water quality volume surface area including the internal berm but excluding the freeboard, $L_{wq\text{-}tot}$:

$$L_{wq-tot} = L_{tot} - 2Zd_{fb} \quad \text{(Equation E-79)}$$

Where:

Z = interior side slope as length per unit height

d_{fb} = freeboard depth

13) Calculate the width of the water quality volume surface area including the internal berm but excluding freeboard, $W_{wq\text{-}tot}$:

$$W_{wq-tot} = W_{tot} - 2Zd_{fb} \quad \text{(Equation E-80)}$$

14) Calculate the total water quality volume surface area including the internal berm and excluding freeboard, $A_{wq\text{-}tot}$:

$$A_{wq-tot} = L_{wq-tot} \times W_{wq-tot} \quad \text{(Equation E-81)}$$

15) Calculate the area of the berm, A_{berm}:

$$A_{berm} = W_{berm} \times L_{berm} \quad \text{(Equation E-82)}$$

Where:

W_{berm} = width of the internal berm

L_{berm} = length of the internal berm

16) Calculate the water quality surface area excluding the internal berm and freeboard, A_{wq}:

$$A_{wq} = A_{wq=tot} - A_{berm} \quad \text{(Equation E-83)}$$

Step 3: Determine Dimensions of Forebay

30-50% of the SQDV is required to be within the active volume of forebay.

1) Calculate the active volume of forebay, V_1:

$$V_1 = \frac{SQDV \times \%V_1}{100} \quad \text{(Equation E-84)}$$

Where:

$\%V_1$ = percent of SQDV in forebay (%)

2) Calculate the surface area for the active volume of forebay, A_1:

$$A_1 = \frac{V_1}{d_1} \quad \text{(Equation E-85)}$$

Where:

d_1 = average depth fo rhte active volume of forebay (2 -4 ft) (ft)

3) Calculate the length of forebay, L_1. Note, inlet and outlet should be configured to maximize the residence time.

$$L_1 = \frac{A_1}{W_1} \qquad \text{(Equation E-86)}$$

Where:

W_1 = width of forebay (ft), $W_1 = W_{av\text{-}tot} = L_{berm}$

Step 4: Determine Dimensions of Cell 2

Cell 2 will consist of the remainder of the basin's active volume.

1) Calculate the active volume of Cell 2, V_2:

$$V_2 = SQDV - V_1 \qquad \text{(Equation E-87)}$$

2) Calculate the surface area of Cell 2, A_2:

$$A_2 = A_{wq} - A_1 \qquad \text{(Equation E-88)}$$

3) Calculate the top length of Cell 2, L_2:

$$L_2 = \frac{A_2}{W_2} \qquad \text{(Equation E-89)}$$

Where:

W_2 = width of Cell 2 (ft), $W_2 = W_1 = W_{wq\text{-}tot} = L_{berm}$

4) Verify that the length-to-width ratio of Cell 2, LW_2, is at least 3:1 with ≥ 4:1 preferred. If the length-to-width ratio is less than 3:1, modify input parameters until a ratio of at least 3:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the pond should be chosen.

$$LW_2 = \frac{L_2}{W_2} \qquad \text{(Equation E-90)}$$

5) Calculate the very shallow zone surface area, A_{vs}:

$$A_{vs} = \frac{A_2 \bullet \%A_{vs}}{100} \qquad \text{(Equation E-91)}$$

Where:

$\%A_{vs}$ = percent of surface area of very shallow zone

6) Calculate the volume of the shallow zone, V_{vs}:

$$V_{vs} = A_{vs} \bullet d_{vs} \qquad \text{(Equation E-92)}$$

Where:

d_{vs} = average depth of the very shallow zone (0.1 – 1 ft)

7) Calculate the length of the very shallow zone, L_{vs}:

$$L_{vs} = \frac{A_{vs}}{W_{vs}} \qquad \text{(Equation E-93)}$$

Where:

W_{vs} = width of the very shallow zone (ft), $W_{vs} = W_2$

8) Calculate the surface area of the shallow zone, A_s:

$$A_s = \frac{A_2 \bullet \%A_s}{100} \qquad \text{(Equation E-94)}$$

Where:

$\%A_s$ = percent of surface area of shallow zone

9) Calculate the volume of the shallow zone, V_s:

$$V_s = A_s \bullet d_s \qquad \text{(Equation E-95)}$$

Where:

d_s = average depth of shallow zone (1 - 3 ft)

10) Calculate length of the shallow zone, L_s:

$$L_s = \frac{A_s}{W_s} \qquad \text{(Equation E-96)}$$

Where:

W_s = width of the shallow zone (ft), $W_s = W_2$

11) Calculate the surface area of the deep zone, A_{deep}:

$$A_{deep} = A_2 - A_{vs} - A_s \qquad \text{(Equation E-97)}$$

12) Calculate the volume of the deep zone, V_{deep}:

$$V_{deep} = V_2 - V_{vs} - V_s \qquad \text{(Equation E-98)}$$

13) Calculate the average depth of the deep zone (3-5 ft), d_{deep}:

$$d_{deep} = \frac{V_{deep}}{A_{deep}} \quad \text{(Equation E-99)}$$

14) Calculate length of the deep zone, L_{deep}:

$$L_{deep} = \frac{A_{deep}}{W_{deep}} \quad \text{(Equation E-100)}$$

Where:

W_{deep} = width of the deep zone (ft), $W_{deep} = W_2$

Step 5: Ensure design requirements and site constraints are achieved

Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the basin is inadequate to meet the design requirements, choose a new location or alternative treatment BMP.

Step 6: Size Outlet Structure

For wetlands with detention, the outlet structures shall be designed to provide 12 hours emptying time for the water quality volume or the required detention necessary for achieving the peak runoff discharge requirements if the extended detention is designed for flow attenuation.

The wetland outlet pipe shall be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for on-line basins or flows greater than the peak runoff discharge rate for the 100-year, 24-hr design storm for on-line basins.

Step 7: Determine Emergency Spillway Requirements

For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm in order to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the 100-yr, 24-hr post-development peak storm water runoff discharge rate directly to the downstream conveyance system or another acceptable discharge point. For sites where the emergency spillway discharges to a steep slope, an emergency overflow riser, in addition to the spillway should be provided.

Sizing Worksheet

Step 1: Determine water quality design volume		
1-1. Enter drainage area, A	A =	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	%
1-3. Determine the maximum allowed effective impervious area, $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	
1-5. Determine the Project Total Impervious area, $TIA=A_{project}*Imp$	TIA=	acres
1-6. Determine the total area from which runoff must be retained, $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\ (1-imp)$	C =	
1-9. Enter design rainfall depth of the storm, P_i (in)	P_i =	in
1-10. Calculate rainfall depth, $P = P_i/12$	P =	ft
1-11. Calculate water quality design volume, $SQDV = 43560 \bullet P*A_{retain}*C$	SQDV =	ft^3
Step 2: Determine Wetland Location, Wetland Type and Preliminary Geometry Based on Site Constraints		
2-1. Based on site constraints, determine the wetland geometry and the storage available by developing an elevation-storage relationship for the wetland. For this simple example, assume a trapezoidal geometry for cell 1 (forebay) and cell 2. The wetland does not have extended detention.		

2-2. Enter the total surface area of the wetland footprint based on site constraints, A_{tot}	A_{tot} = ft^2
2-3. Enter the length of the wetland footprint based on site constraints, L_{tot}	L_{tot} = ft
2-4. Calculate the width of the wetland footprint, $W_{tot} = A_{tot} / L_{tot}$	W_{tot} = ft
2-5. Enter interior side slope as length per unit height (min = 3), Z	Z =
2-6. Enter desired freeboard depth, d_{fb}	d_{fb} = ft
2-7. Calculate the length of the water quality volume surface area including the internal berm but excluding freeboard, $L_{wq\text{-}tot} = L_{tot} - 2Zd_{fb}$	$L_{wq\text{-}tot}$ = ft
2-8. Calculate the width of the water quality volume surface area including the internal berm but excluding freeboard, $W_{wq\text{-}tot} = W_{tot} - 2Zd_{fb}$	$W_{wq\text{-}tot}$ = ft
2-9. Calculate the total water quality volume surface area including the internal berm and excluding freeboard, $A_{wq\text{-}tot} = L_{wq\text{-}tot} \bullet W_{wq\text{-}tot}$	$A_{wq\text{-}tot}$ = ft^2
2-10. Enter the width of the internal berm (6 ft min), W_{berm}	W_{berm} = ft
2-11. Enter the length of the internal berm, $L_{berm} = W_{wq\text{-}tot}$	L_{berm} = ft
2-12. Calculate the area of the berm, $A_{berm} = W_{berm} \bullet L_{berm}$	A_{berm} = ft^2
2-13. Calculate the water quality volume surface area excluding the internal berm and freeboard, $A_{wq} = A_{wq\text{-}tot} - A_{berm}$	A_{wq} = ft^2
Step 3: Determine Dimensions of forebay	
3-1. Enter the percent of SQDV in forebay (30-50% required), $\%V_1$	$\%V_1$ = %
3-2. Calculate the active volume of forebay (includes water quality volume + sediment storage volume),	V_1 = ft^3

$V_1 = (SQDV \bullet \%V_1)\ /100$	
3-3. Enter desired average depth of forebay1 (2-4 ft including sediment storage of 1 ft), d_1	d_1 = ft
3-4. Calculate the surface area for the water quality volume of forebay, $A_1 = V_1 / d_1$	A_1 = ft^2
3-5. Enter the width of forebay, $W_1 = W_{av\text{-}tot} = L_{berm}$	W_1 = ft
3-6. Calculate the length of forebay (Note: inlet and outlet should be configured to maximize the residence time), $L_1 = A_1 / W1$	L_1 = ft
Step 4: Determine Dimensions of Cell 2	
4-1. Calculate the active volume of Cell 2, $V_2 = SQDV - V_1$	V_2 = ft^3
4-2. Calculate surface area of Cell 2, $A_2 = A_{wq} - A_1$	A_2 = ft^2
4-3. Enter width of Cell 2, $W_2 = W_1 = W_{wq\text{-}tot} = L_{berm}$	W_2 = ft
4-4. Calculate top length of Cell 2, $L_2 = A_2 / W_2$	L_2 = ft
4-5. Verify that the length-to-width ratio of Cell 2 is at least 3:1 with ≥ 4:1 preferred. If the length-to-width ratio is less than 3:1, modify input parameters until a ratio of at least 3:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the pond should be chosen, $LW_2 = L_2 / W_2$	LW_2 =
4-6. Enter percent of surface area of very shallow zone, $\%A_{vs}$	$\%A_{vs}$ = %
4-7. Calculate very shallow zone surface area, $A_{vs} = (A_2 \bullet \%A_{vs})/100$	A_{vs} = ft^2
4-8. Enter average depth of very shallow zone (0.1 - 1 ft), d_{vs}	d_{vs} = ft
4-9. Calculate volume of very shallow zone, $V_{vs} = A_{vs} \bullet d_{vs}$	V_{vs} = ft^3
4-10. Enter width of very shallow zone, $W_{vs} = W_2$	W_{vs} = ft

4-11. Calculate length of very shallow zone, $L_{vs} = A_{vs} / W_{vs}$	L_{vs} = ft
4-12. Enter percent of surface area of shallow zone, $\%A_s$	$\%A_s$ = %
4-13. Calculate surface area of shallow zone, $A_s = (A_2 \bullet \%A_s)/100$	A_s = ft^2
4-14. Enter average depth of shallow zone (1 - 3 ft), d_s	d_s = ft
4-15. Calculate volume of shallow zone, $V_s = A_s \bullet d_s$	V_s = ft^3
4-16. Enter width of shallow zone, $W_s = W_2$	W_s = ft
4-17. Calculate length of shallow zone, $L_s = A_s / W_s$	L_s = ft
4-18. Calculate surface area of deep zone, $A_{deep} = A_2 - A_{vs} - A_s$	A_{deep} = ft^2
4-19. Calculate volume of deep zone, $V_{deep} = V_2 - V_{vs} - V_s$	V_{deep} = ft^3
4-20. Calculate average depth of deep zone (3 - 5 ft), $d_{deep} = V_{deep} / A_{deep}$	d_{deep} = ft
4-21. Enter width of deep zone, $W_{deep} = W_2$	W_{deep} = ft
4-22. Calculate length of deep zone, $L_{deep} = A_{deep} / W_{deeo}$	L_{deep} = ft
Step 5: Ensure Design Requirements and Site Constraints are Achieved	
5-1. Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the wetland is inadequate to meet the design requirements, choose a new location for the wetland or select an alternative treatment BMP.	

Step 6: Size Outlet Structure
6-1. The wetland outlet pipe shall be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for off-line basins or flow from the capital storm for on-line basins.
Step 7: Determine Emergency Spillway Requirements
7-1. For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm in order to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the 100-yr, 24-hr post-development peak storm water runoff discharge rate directly to the downstream conveyance system or another acceptable discharge point.

Design Example

Wetland siting requires the following considerations prior to construction: (1) availability of base flow – stormwater wetlands require a regular source of water to support wetland biota, (2) slope stability – stormwater wetlands are not permitted near steep slope hazard areas, (3) surface space availability – large footprint area is required, and (4) compatibility with flood control – basins must not interfere with flood control functions of existing conveyance and detention structures.

The wetland in this example does not have extended detention. An internal berm separates the forebay (Cell 1) and the main basin (Cell 2). The berm is at the elevation of the active volume (SQDV plus sediment storage volume) design surface which is also the permanent wetpool elevation.

Step 1: Determine Water Quality Design Volume

For this design example, a 20-acre residential development with a 60% total impervious area is considered. The 85th percentile storm event for the project location is 0.75 inches.

Step 1: Determine water quality design volume			
1-1. Enter drainage area, A	A =	20	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	5	
1-3. Determine the maximum allowed effective impervious area, $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	1.0	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	0.6	
1-5. Determine the Project Total Impervious area, $TIA=A_{project}*Imp$	TIA=	12	acres
1-6. Determine the total area from which runoff must be retained, $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	11	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	0.05	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\ (1\text{-}imp)$	C =	0.59	
1-9. Enter design rainfall depth of the storm, P_i (in)	P_i =	0.75	in

1-10. Calculate rainfall depth, $P = P_i/12$	P = 0.06 ft
1-11. Calculate water quality design volume, $SQDV = 43560 \bullet P * A_{retain} * C$	SQDV = 17,000 ft^3

Step 2: Determine Pond Location and Preliminary Geometry Based on Site Constraints

A total footprint area and total length available for the wetland is provided. This step calculates the total active volume surface area which is equivalent to the permanent wetpool surface area. This step also calculates the dimensions of the internal berm.

Step 2: Determine Wetland Location, Wetland Type and Preliminary Geometry Based on Site Constraints	
2-1. Based on site constraints, determine the wetland geometry and the storage available by developing an elevation-storage relationship for the wetland. For this simple example, assume a trapezoidal geometry for cell 1 (forebay) and cell 2. The wetland does not have extended detention.	
2-2. Enter the total surface area of the wetland footprint based on site constraints, A_{tot}	A_{tot} = 7,500 ft^2
2-3. Enter the length of the wetland footprint based on site constraints, L_{tot}	L_{tot} = 200 ft
2-4. Calculate the width of the wetland footprint, $W_{tot} = A_{tot} / L_{tot}$	W_{tot} = 38 ft
2-5. Enter interior side slope as length per unit height (min = 3), Z	Z = 3
2-6. Enter desired freeboard depth, d_{fb}	d_{fb} = 2 ft
2-7. Calculate the length of the water quality volume surface area including the internal berm but excluding freeboard, $L_{wq\text{-}tot} = L_{tot} - 2Zd_{fb}$	$L_{wq\text{-}tot}$ = 188 ft
2-8. Calculate the width of the water quality volume surface area including the internal berm but excluding freeboard, $W_{wq\text{-}tot} = W_{tot} - 2Zd_{fb}$	$W_{wq\text{-}tot}$ = 26 ft

Step 2: Determine Wetland Location, Wetland Type and Preliminary Geometry Based on Site Constraints			
2-9. Calculate the total water quality volume surface area including the internal berm and excluding freeboard, $A_{wq\text{-}tot} = L_{wq\text{-}tot} \bullet W_{wq\text{-}tot}$	$A_{wq\text{-}tot}$ =	4,900	ft^2
2-10. Enter the width of the internal berm (6 ft min), W_{berm}	W_{berm} =	6	ft
2-11. Enter the length of the internal berm, $L_{berm} = W_{wq\text{-}tot}$	L_{berm} =	26	ft
2-12. Calculate the area of the berm, $A_{berm} = W_{berm} \bullet L_{berm}$	A_{berm} =	160	ft^2
2-13. Calculate the active volume surface area excluding the internal berm and freeboard, $A_{wq} = A_{wq\text{-}tot} - A_{berm}$	A_{wq} =	4,740	ft^2

Step 3: Determine Dimensions of Forebay

It should be assumed that the forebay should be 30-50% of the SQDV.

Step 3: Determine Dimensions of forebay			
3-1. Enter the percent of SQDV in forebay (30-50% required), $\%V_1$	$\%V_1$ =	30	%
3-2. Calculate the active volume of forebay (including sediment storage), $V_1 = (SQDV \bullet \%V_1)/100$	V_1 =	5,100	ft^3
3-3. Enter desired average depth of forebay (2-4 ft including sediment storage of 1 ft), d_1	d_1 =	4	ft
3-4. Calculate the surface area for the water quality volume of forebay, $A_1 = V_1 / d_1$	A_1 =	1,275	ft^2
3-5. Enter the width of forebay, $W_1 = W_{av\text{-}tot} = L_{berm}$	W_1 =	38	ft
3-6. Calculate the length of forebay (<u>Note:</u> inlet and outlet should be configured to maximize the residence time), $L_1 = A_1 / W1$	L_1 =	34	ft

Step 4: Determine Dimensions of Cell 2

Verify that the surface area and length-to-width ratio of Cell 2 meet the design criteria. Calculate volumes, depths and surface areas for the very shallow, shallow and deep zones.

Step 4: Determine Dimensions of Cell 2			
4-1. Calculate the active volume of Cell 2, V_2 = SQDV - V_1	V_2 =	11,900	ft^3
4-2. Calculate surface area of Cell 2, $A_2 = A_{wq} - A_1$	A_2 =	3,460	ft^2
4-3. Enter width of Cell 2, $W_2 = W_1 = W_{wq\text{-}tot} = L_{berm}$	W_2 =	26	ft
4-4. Calculate top length of Cell 2, $L_2 = A_2 / W_2$	L_2 =	130	ft
4-5. Verify that the length-to-width ratio of Cell 2 is at least 3:1 with ≥ 4:1 preferred. If the length-to-width ratio is less than 3:1, modify input parameters until a ratio of at least 3:1 is achieved. If the input parameters cannot be modified as a result of site constraints, another site for the pond should be chosen, $LW_2 = L_2 / W_2$	LW_2 =	5	
4-6. Enter percent of surface area of very shallow zone, $\%A_{vs}$	$\%A_{vs}$ =	15	ft^2
4-7. Calculate very shallow zone surface area, $A_{vs} = (A_2 \bullet \%A_{vs})/100$	A_{vs} =	520	ft^2
4-8. Enter average depth of very shallow zone (0.1 - 1 ft), d_{vs}	d_{vs} =	1	ft
4-9. Calculate volume of very shallow zone, $V_{vs} = A_{vs} \bullet d_{vs}$	V_{vs} =	520	ft^3
4-10. Enter width of very shallow zone, $W_{vs} = W_2$	W_{vs} =	26	ft
4-11. Calculate length of very shallow zone, $L_{vs} = A_{vs} / W_{vs}$	L_{vs} =	20	ft
4-12. Enter percent of surface area of shallow zone, $\%A_s$	$\%A_s$ =	55	
4-13. Calculate surface area of shallow zone, $A_s = (A_2 \bullet \%A_s)/100$	A_s =	1,900	ft^2
4-14. Enter average depth of shallow zone (1 - 3 ft), d_s	d_s =	3	ft
4-15. Calculate volume of shallow zone, $V_s = A_s \bullet d_s$	V_s =	5,700	ft^3
4-16. Enter width of shallow zone, $W_s = W_2$	W_s =	26	ft
4-17. Calculate length of shallow zone, $L_s = A_s / W_s$	L_s =	220	ft

Step 4: Determine Dimensions of Cell 2			
4-18. Calculate surface area of deep zone, $A_{deep} = A_2 - A_{vs} - A_s$	A_{deep} =	1,040	ft^2
4-19. Calculate volume of deep zone, $V_{deep} = V_2 - V_{vs} - V_s$	V_{deep} =	5,680	ft^3
4-20. Calculate average depth of deep zone (3 - 5 ft), $d_{deep} = V_{deep} / A_{deep}$	d_{deep} =	5	ft
4-21. Enter width of deep zone, $W_{deep} = W_2$	W_{deep} =	26	ft
4-22. Calculate length of deep zone, $L_{deep} = A_{deep} / W_{deeo}$	L_{deep} =	40	ft

Step 5: Ensure Design Requirements and Site Conditions are Achieved

Check design requirements and site constraints. Modify design geometry until requirements are met. If the chosen site for the wetland is inadequate to meet the design requirements, choose a new location for the wetland or select an alternative treatment BMP.

Step 6: Size Outlet Structure

6-1. The wetland outlet pipe shall be sized, at a minimum, to pass flows greater than the stormwater quality design peak flow for off-line basins or flow from the capital storm for on-line basins.

Step 7: Determine Emergency Spillway Requirements

For online basins, an emergency overflow spillway should be sized to pass flows greater than the design peak runoff discharge rate for the 100-yr, 24-hr storm in order to prevent overtopping of the walls or berms in the event that a blockage of the riser occurs. For offline basins, an emergency spillway or riser should be sized to pass the 100-yr, 24-hr post-development peak storm water runoff discharge rate directly to the downstream conveyance system or another acceptable discharge point.

E.11 TCM-4 Sand Filters

Sizing Methodology

A sand filter is designed with two parts: (1) a temporary storage reservoir to store runoff, and (2) a sand filter bed through which the stored runoff must percolate. Usually the storage reservoir is simply placed directly above the filter, and the floor of the reservoir pond is the top of the sand bed. For this case, the storage volume also determines the hydraulic head over the filter surface, which increases the rate of flow through the sand.

Two methods are available for sizing sand filters: a simple method and a routing modeling method. The simple method uses standard values to define filter hydraulic characteristics for determining the sand surface area. This method is useful for planning purposes, for a first approximation to begin iterations in the detailed method, or when use of the detailed computer model is not desired or not available. The simple method very often results in a larger filter than the routing method.

Background

Sand filter design is based on Darcy's law:

$$Q = KiA \qquad \text{(Equation E-101)}$$

Where:

Q	=	water quality design flow (cfs)
K	=	hydraulic conductivity (fps)
A	=	surface area perpendicular to the direction of flow (ft^2)
i	=	hydraulic gradient (ft/ft) for a constant head and constant media depth, computed as follows:

$$i = \frac{h+l}{l} \qquad \text{(Equation E-102)}$$

Where:

h	=	average depth of water above the filter (ft), defined for this design as $d/2$
d	=	maximum storage depth above the filter (ft)
l	=	thickness of sand media (ft)

Darcy's law underlies both the simple and the routing methods of design. The filtration rate V, or more correctly, $1/V$, is the direct input in the sand filter design. The relationship between the filtration rate V and hydraulic conductivity K is revealed by equating Darcy's law and the equation of continuity, $Q = VA$. Specifically:

$Q = KiA$ and $Q = VA$

So, $VA = KiA$

Or: $V = Ki$ (Equation E-103)

Where,

V = filtration rate (ft/s)

Note that $V \neq K$. That is, the filtration rate is not the same as the hydraulic conductivity, but they do have the same units (distance per time). K can be equated to V by dividing V by the hydraulic gradient i, which is defined above.

The hydraulic conductivity K does not change with head nor is it dependent on the thickness of the media, only on the characteristics of the media and the fluid. A design hydraulic conductivity of 1 inch per hour (2 feet per day) used in this simple sizing method is based on bench-scale tests of conditioned rather than clean sand (KCSWDM, 2005) and represents the average sand bed condition as silt is captured and held in the sand bed.

Unlike the hydraulic conductivity, the filtration rate V changes with head and media thickness, although the media thickness is constant in the sand filter design.

Simple Sizing Method

The simple sizing method does not route flows through the filter. It determines the size of the filter based on the simple assumption that inflow is immediately discharged through the filter as if there were no storage volume. An adjustment factor (0.7) is applied to compensate for the greater filter size resulting from this method. Even with the adjustment factor, the simple method generally produces a larger filter size than the routing method.

Step 1: Determine the water quality design volume

Sand filters should be sized to capture and treat the stormwater quality design volume (see Section E.1).

Step 2: Determine maximum storage depth of water

Determine the maximum water storage depth (d) above the sand filter. This depth is defined as the depth at which water begins to overflow the reservoir pond, and it depends on the site topography and hydraulic constraints. The depth is chosen by the designer, but shall be 6 feet or less.

Step 3: Calculate the sand filter area

Determine the sand filter area using the following equation:

$$A_{sf} = \frac{V_{wq} R L}{K t (h + L)} \qquad \text{(Equation E-104)}$$

Where,

A_{sf}	=	surface area of the sand filter bed (ft^2)
V_{wq}	=	water quality design volume (ft^3)
R	=	routing adjustment factor (use R = 0.7)
L	=	sand bed depth (ft)
K	=	design hydraulic conductivity (use 2 ft/day)
t	=	drawdown time (use 1 day)
h	=	average depth of water above the filter (ft), (use $d/2$ with d from Step 1)

Routing Method

A continuous runoff model, such as US EPA's Storm Water Management Model (SWMM) Model, can be used to optimally size a sand filter. A continuous simulation model consists of three components: a representative long term period of rainfall data ($\approx$ 20 years or greater) as the primary model input; a model component representing the tributary area to the sand filter that takes into account the amount of impervious area, soil types of the pervious area, vegetation, evapotranspiration, etc.; and a component that simulates the sand filter. Using this method, the filter should be sized to capture and treat the WQ design volume from the post-development tributary area.

The continuous simulation model routes predicted tributary runoff to the sand filter, where treatment is simulated as a function of the infiltrative (flow) capacity of the sand filter and the available storage volume above the sand filter. In a continuous runoff model such as SWMM, the physical parameters of the sand filter are represented with stage-storage-discharge relationships. Due to the computational power of ordinary desktop computers, long-term continuous simulations generally take only minutes to run. This allows the modeler to run several simulations for a range of sand filter sizes, varying either the surface area of the filter (and resulting flow capacity) or the storage capacity above the sand filter, or both. Sufficient continuous model simulations should be completed so that results encompass the WQ design volume capture goal.

Model results should be plotted for both varying storage depths above the filter and for varying filter surface area (and resulting flow capacity) while keeping all other parameters constant. The resulting relationship of percent capture as a function of sand filter flow and storage capacity can be used to optimally size a sand filter based on site conditions and restraints.

In addition to continuous simulation modeling, routing spreadsheets and/or other forms of routing modeling that incorporate rainfall-runoff relationships and infiltrative (flow) capacities of sand filters may be used to size facilities. Alternative sizing methodologies should be prepared with good engineering practices.

Sizing Worksheet

Step 1: Determine water quality design volume	
1-1. Enter Project area (acres), $A_{project}$	$A_{project}$ = acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ = %
1-3. Determine the maximum allowed effective impervious area (ac), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$= acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA= acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} = acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p\,(1-imp)$	C =
1-9. Enter design rainfall depth of the storm (in), P_i	P_i = in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P = ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560\bullet C*P*A_{retain}$	SQDV= ac-ft
Step 2: Determine maximum storage depth of water	
2-1. Determine the maximum storage depth (max 6 ft) of water above the sand filter, *d* (ft)	*d* = ft

Step 3: Calculate sand filter area	
3-1. Enter water quality design volume, *SQDV*	SQDV = ft³
3-2. Enter routing adjustment factor (use R =0.7), R	R =
3-3. Enter thickness of sand filter (min. 2 ft, 3 ft preferred), L	L = ft
3-4. Enter design hydraulic conductivity of media (use 2 ft/day), K_{des}	K = ft/day
3-5. Enter drawdown time, t	t = day
3-6. Calculate average depth of water above the filter, $h = d/2$	h = ft
3-7. Calculate sand filter area, $A_{sf} = (SQDV*RL)/(Kt\ (h+L))$	A_{sf} = ft²
Step 4: Determine filter dimensions	
4-1. Sand filter area, A_{sf}	A_{sf} = ft²
4-2. Enter geometric configuration, LR:W ratio (2:1 or greater), L_R	L_R =
4-3. Select the width of the sand filter, W	W = ft
4-4. Calculate the length of the sand filter, $L=WL_R$	L = ft
4-5. Calculate rate of filtration, $r_{wq} = Ki$; where $i = \frac{h+l}{l}$	r_{wq} = ft/d
Step 5: Calculate filter longitudinal underdrain collection pipe	
5-1. Calculated filtered flow rate, $Q_f = r_{wq}A_{sf}/86400$	Q_f = cfs
5-2. Enter minimum slope for energy gradient, S_e	S_e =

5-3. Enter Hazen-Williams coefficient for plastic, *C*	C =
5-4. Enter pipe diameter (6" min.), *D*	D = in
5-5. Calculate pipe hydraulic radius, $R_h = D/48$	R_h = ft
5-6. Calculate velocity at the outlet of the pipe, $V_p = 1.318CR_h0.63S_e0.54$	V_p = ft/s
5-7. Calculate pipe capacity, $Q_{cap} = 0.25\pi(D/12)^2V_p$	Q_{cap} = cfs
Step 7: Provide conveyance capacity for filter clogging	
7-1. The sand filters should be placed off-line, but an emergency overflow must still be provided in the event the filter becomes clogged.	

Design Example

Step 1: Determine water quality design volume

For this design example, a 10-acre site with soil type 4 and 60% total impervious area is considered. The 85th percentile storm event for the project location is 0.75 inches.

Step 1: Determine water quality design volume			
1-1. Enter Project area (acres), $A_{project}$	$A_{project}$ =	10	acres
1-2. Enter the maximum allowable percent of the Project area that may be effective impervious area (refer to permit), ranges from 5-30%, $\%_{allowable}$	$\%_{allowable}$ =	5	
1-3. Determine the maximum allowed effective impervious area (ac), $EIA_{allowable} = (A_{project})*(\%_{allowable})$	$EIA_{allowable}$=	0.5	acres
1-4. Enter Project impervious fraction, *Imp* (e.g. 60% = 0.60)	Imp=	0.6	
1-5. Determine the Project Total Impervious area (acres), $TIA=A_{project}*Imp$	TIA=	6	acres
1-6. Determine the total area from which runoff must be retained (acres), $A_{retain}=TIA-EIA_{allowable}$	A_{retain} =	5.5	acres
1-7. Determine pervious runoff coefficient using Table E-1, C_p	C_p =	0.05	
1-8. Calculate runoff coefficient, $C = 0.95*imp + C_p (1-imp)$	C =	0.59	
1-9. Enter design rainfall depth of the storm (in), P_i	P_i =	0.75	in
1-10. Calculate rainfall depth (ft), $P = P_i/12$	P =	0.06	ft
1-11. Calculate water quality design volume (ft^3), $SQDV=43560 \bullet C*P*A_{retain}$	SQDV=	0.20	ac-ft

Step 1a: Determine maximum storage depth of water

Determine the maximum storage depth of water above the sand filter.

Step 1a: Determine maximum storage depth of water			
1a-1. Determine the maximum storage depth (max 6 ft) of water above the sand filter, *d* (ft)	*d* =	6	ft

Step 2: Calculate Sand Filter Area

A sand filter is designed with two components: (1) temporary storage reservoir to store runoff, and (2) a sand filter bed through which the stored runoff must percolate getting treatment.

The simple sizing method does not rout flows through the filter. The size of the filter is determined based on the simple assumption that inflow is immediately discharged through the filter. The adjustment factor, R, is applied to compensate for the greater filter size resulting from this method.

Step 2: Calculate sand filter area			
2-1. Enter water quality design volume, *SQDV*	SQDV =	o.20	ac-ft
2-2. Enter routing adjustment factor (use R =0.7), *R*	*R* =	0.7	
2-3. Enter thickness of sand filter (min. 2 ft, 3 ft preferred), *L*	*L* =	2	ft
2-4. Enter design hydraulic conductivity (use 2 ft/day), *K*	*K* =	2	ft/day
2-5. Enter drawdown time (use 1 day), *t*	*t* =	2	day
2-6. Calculate average depth of water above the filter, $h = d/2$	h =	3	ft
2-7. Calculate sand filter area, $A_{sf} = (SQDV*RL)/(Kt\,(h+L))$	A_{sf} =	0.014	acre

Step 3: Determine Filter Dimensions

Step 3: Determine filter dimensions			
3-1. Sand filter area in ft², $A_{sf(feet)}=A_{sf(acre)}*43,560$	A_{sf} =	610	ft²
3-2. Enter geometric configuration, LR:W ratio (2:1 min.), L_R	L_R =	2	
3-3. Calculate the width of the sand filter, *W*	W =	18	ft

Step 3: Determine filter dimensions			
3-4. Calculate the length of the sand filter, L	L =	36	ft
3-5. Calculate rate of filtration, $r_{wq} = Ki$, where $i = \frac{h+l}{l}$	r_{wq} =	2.3	ft/d

Step 4: Calculate Filter Longitudinal Underdrain Collection Pipe

All underdrain pipes must be 6 inches or greater to facilitate cleaning.

Step 5: Calculate filter longitudinal underdrain collection pipe			
5-1. Calculated filtered flow rate, $Q_f = r_{wq}A_{sf}/86400$	Q_f =	0.01	cfs
5-2. Enter minimum slope for energy gradient, Se	S_e =	0.005	
5-3. Enter Hazen-Williams coefficient for plastic, C	C =	140	
5-4. Enter pipe diameter (6" min), D	D =	6	in
5-5. Calculate pipe hydraulic radius, $R_h = D/48$	R_h =	0.13	
5-6. Calculate velocity at the outlet of the pipe, $V_p = 1.318CR_h^{0.63}S_e^{0.54}$	V_p =	2.9	ft/s
5-7. Calculate pipe capacity, $Q_{cap} = 0.25\pi(D/12)^2V_p$	Q_{cap} =	0.57	cfs

Step 5: Provide Conveyance Capacity for Filter Clogging

The sand filters should be placed off-line, but an emergency overflow must still be provided in the event the filter becomes clogged.

APPENDIX F : FLOW SPLITTER DESIGN SPECIFICATIONS

F.1 Flow Splitter Introduction

Flow splitters must be provided for off-line facilities to divert the water quality design flow to the BMP and bypass higher flows. In most cases, it is a designer's choice whether storm water treatment BMPs described in this manual are designed as on-line or off-line; exceptions are vegetated strip filters, permeable pavement, and building BMPs which are designed on-line.

A crucial factor in designing flow splitters is to ensure that low flows are delivered to the treatment facility up to the water quality design flow rate. Above this rate, additional flows remain in the storm drain or are diverted to a bypass drain with minimal increase in head at the flow splitter structure to avoid surcharging the water quality facility under high flow conditions.

Flow splitters are typically manholes or vaults with baffles. In place of baffles, the splitter mechanism may be a half tee section with a solid top and an orifice in the bottom of the tee section. A full tee option may also be used (see "Design Criteria" below). Two possible design options for flow splitters are shown in the figures in this Appendix. Other equivalent designs that achieve the result of splitting low flows, up to the WQ design flow, into the WQ treatment facility and divert higher flows around the facility are also acceptable.

Flow splitters may be modeled using standard level pool routing techniques, as described in the Handbook of Applied Hydrology (Ven te Chow; 1964) and elsewhere. The stage/discharge relationship of the outflow pipes shall be determined using backwater analysis techniques. Weirs shall be analyzed as sharp-crested weirs.

Design Criteria

1) A flow splitter shall be designed to deliver the required water quality design flow rate to the storm water treatment facility.

17) The top of the weir shall be located at the water surface for the design flow. Remaining flows enter the bypass line.

18) The maximum head shall be minimized for flow in excess of the water quality design flow. Specifically, flow to the treatment facility at the flood control design storm water surface shall not increase the design water quality design flow by more than 10%.

19) Example designs are shown in the figures in this Appendix. Equivalent designs are also acceptable.

20) Special applications, such as roads, may require the use of a modified flow splitter. The baffle wall may be fitted with a notch and adjustable weir plate to proportion runoff volumes other than high flows.

21) For ponding facilities, backwater effects must be included in designing the height of the standpipe in the manhole.

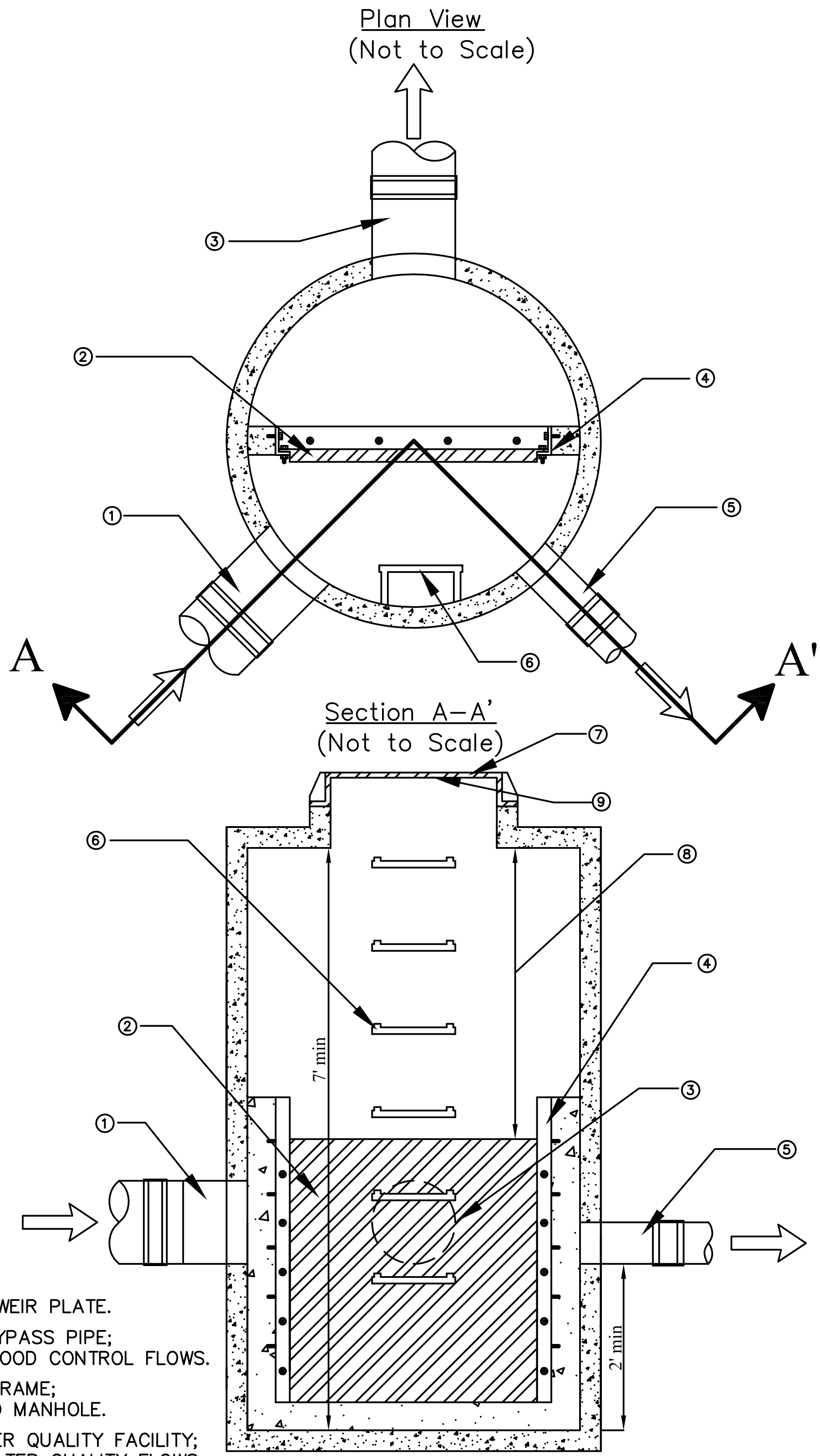
22) Ladder or step and handhold access shall be provided. If the weir wall is higher than 36 inches, two ladders, on the either side of the wall, are required.

F.2 Material Requirements

1) The splitter baffle shall be installed in a standard manhole or vault. The baffle wall shall be made of material resistant to corrosion (minimum 4-inch thick reinforced concrete, Type 302 or Type 316 stainless steel plate, or equivalent).

23) The minimum clearance between the top of the baffle wall and the bottom of the manhole or vault cover shall be 4 feet; otherwise, dual access points shall be provided.

24) All metal parts shall be corrosion resistant. Examples of preferred materials include aluminum, stainless steel, and plastic. Zinc and galvanized materials are not permitted because of aquatic toxicity. Painting metal parts shall not be allowed because of poor longevity.



NOTES:

① INLET PIPE.
② ADJUSTABLE WEIR PLATE.
③ HIGH FLOW BYPASS PIPE; SIZED FOR FLOOD CONTROL FLOWS.
④ WEIR PLATE FRAME; ANCHORED TO MANHOLE.
⑤ PIPE TO WATER QUALITY FACILITY; SIZED FOR WATER QUALITY FLOWS.
⑥ ACCESS STEPS.
⑦ 24" ROUND FRAME AND SOLID LID.
⑧ 4' MIN DISTANCE OR PROVIDE SEPARATE ACCESS ON BOTH SIDES OF WEIR.
⑨ AFFIX PERMANENT IDENTIFICATION TAG.

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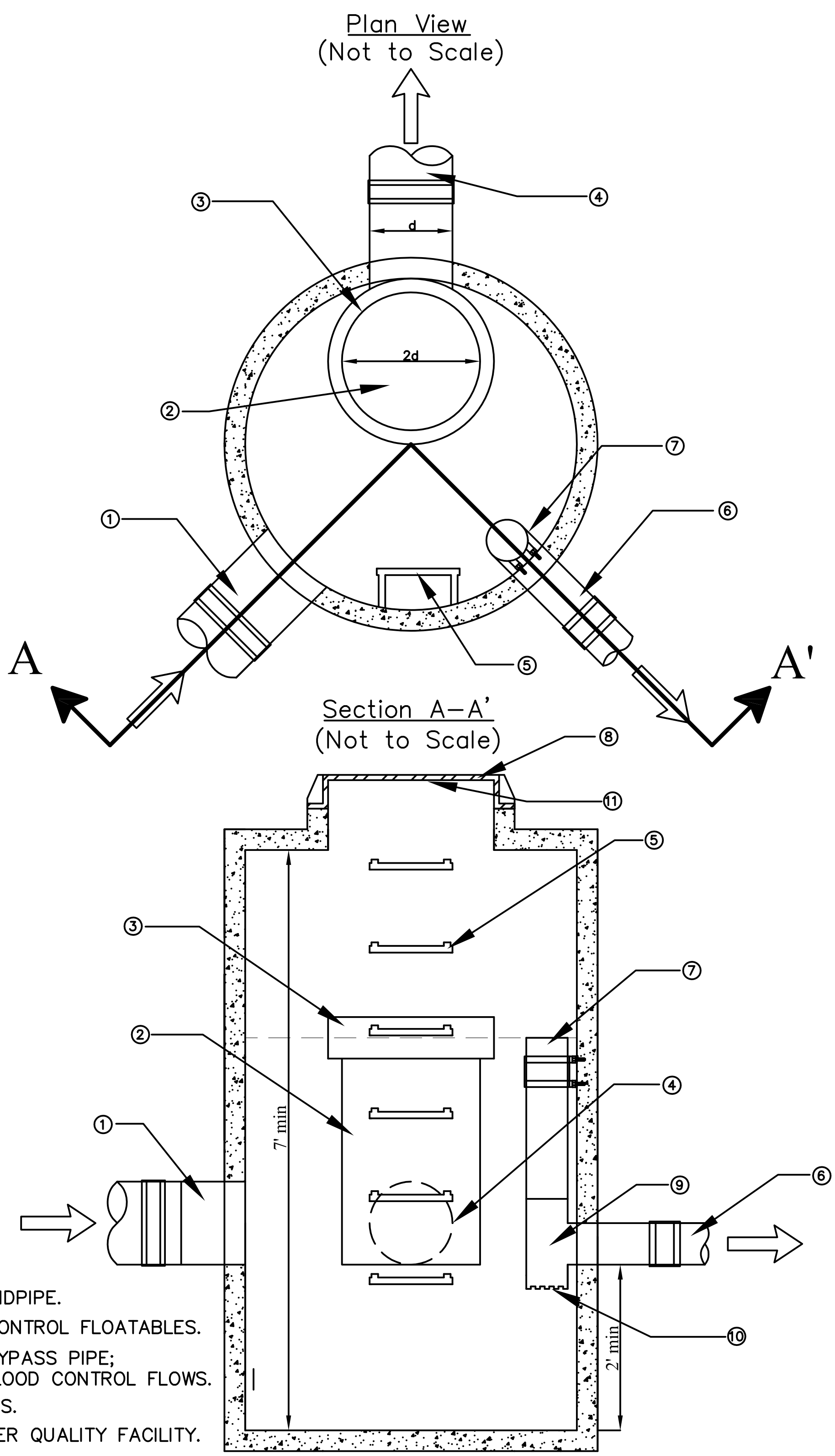
Figure F-1: Flow Splitter Option A



NOTES:

1. INLET PIPE.
2. BYPASS STANDPIPE.
3. BAFFLE TO CONTROL FLOATABLES.
4. HIGH FLOW BYPASS PIPE; SIZED FOR FLOOD CONTROL FLOWS.
5. ACCESS STEPS.
6. PIPE TO WATER QUALITY FACILITY.
7. RISER PIPE; TOP OF PIPE AT DESIGN ELEVATION FOR WATER QUALITY FLOWS.
8. 24" ROUND FRAME AND SOLID LID.
9. REMOVABLE "TEE" SECTION FOR CLEANOUT.
10. RISER PIPE ORIFICE SIZED FOR WATER QUALITY FLOWS.
11. AFFIX PERMANENT IDENTIFICATION TAG.

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Figure F-2: Flow Splitter Option B

APPENDIX G: DESIGN CRITERIA CHECKLISTS FOR STORMWATER RUNOFF BMPS

BIO-1 Bioretention Checklist

- ☐ Has the bioretention facility been sized to treat the water quality design volume, SQDV (see worksheet)?
- ☐ Does the bioretention have a maximum ponding depth of 18 in.?
- ☐ Is the planting soil depth at least 2 feet?
- ☐ Has an underdrain been provided if native soil permeability is less than 0.5 in/hr and infiltration is not possible/allowed?
- ☐ Has a gravel drainage layer been provided if native soil permeability is greater than 0.5 in/hr and infiltration is possible/allowed?
- ☐ Does the bioretention ponding depth drain below the planting soil in less than 48 hours?
- ☐ Is the gravel drainage layer sized to adequately meet the maximum drawdown time of 96 hours?
- ☐ Has the bioretention facility been properly sized as recommended in the manual?
- ☐ Does the flow entrance meet specifications (dispersed, low velocity flow; dispersed flow across pavement; flow spreading trench; cuts or wheel slots for parking lots)?
- ☐ Does the pipe flow entrance include erosion protection material to dissipate flow energy?
- ☐ Is the flow path unblocked by trees and shrubs?
- ☐ Is the underdrain at least 6 inches in diameter?
- ☐ Is the underdrain pipe made of accepted material (slotted PVC pipe conforming to ASTM C 3034 or equivalent HDPE pipe conforming to AASHTO 252M)?
- ☐ Does the slotted pipe have correct sizing and spacing of slots?
- ☐ Is the underdrain sloped at 0.5% or more?
- ☐ Are rigid observation pipes connected to underdrain every 250 to 300 feet of installed pipe?
- ☐ Do the observation pipe wells/clean outs extend 6 inches above top elevation of bioretention facility mulch and are they capped as required?

- ☐ Does the gravel underdrain bedding consist of the correct aggregate?
- ☐ If geotextile fabric is placed between the planting media and gravel layer, does it meet the specifications outlined in the manual?
- ☐ Does the gravel underdrain bedding extend at least 6 inches below the underdrain pipe (if needed) and does it provide 1 foot depth around top and sides of pipe?
- ☐ Does the underdrain drain freely to the accepted discharge point?
- ☐ Is an overflow device consisting of vertical PVC pipe included in design?
- ☐ Has the overflow device been installed at the 18-inch ponding depth?
- ☐ Is the overflow riser at least 6 inches in diameter?
- ☐ Has the inlet to the riser been positioned at least 6 inches above the planting media and capped with a spider cap?
- ☐ If bioretention is close to roads or infrastructure, have infiltration pathways been restricted with geomembrane (at least 30 mm) or clay liners?
- ☐ Is planting soil composed of correct aggregate (60-70% sand; 30-40% compost) and free of stones, stumps and roots?
- ☐ Does compost have acceptable characteristics?
- ☐ Is constructed bioretention facility covered with well-aged mulch, free of seeds, weeds, soil and roots, and at least 2-3 inches thick?
- ☐ Is all bioretention vegetation tolerant of summer drought, ponding fluctuations, and saturated soil conditions for 48 to 72 hours?
- ☐ Have an adequate number of different plant species been incorporated into the bioretention (It is recommended that 3 tree, 3 shrub, and 3 herbaceous groundcover species be included)?
- ☐ Have native plants been used to the maximum extent practicable?

BIO- 2 Planter Box Checklist

- ☐ Is the planter box tributary area less than 15,000 ft2?
- ☐ Is the groundwater level at least 2 feet below the bottom of the planter box?
- ☐ Is there adequate relief between land surface and stormwater conveyance system to permit vertical percolation?
- ☐ Is the planter box located in an area with adequate sunlight to support selected vegetation?
- ☐ Is the planter box sized to treat the water quality design volume, Vwq (see worksheet)?
- ☐ Does the planter box have a maximum ponding depth of 12 inches?
- ☐ Is the planting soil depth at least 2 feet (3 feet preferred)?
- ☐ Does the ponded water drain below the planting soil in less than 48 hours?
- ☐ Has the distance between the downspouts and the overflow outlet been maximized?
- ☐ Has the planter box been sized the same as a Bioretention facility with planter box parameters?
- ☐ Has the planter box been constructed with an appropriate non-leaching permanent material?
- ☐ Has the planter box structure been adequately sealed to ensure that water exits only via the underdrain?
- ☐ Has an underdrain been provided?
- ☐ If the entrance to the planter box is piped, has erosion protection been included in the design (erosion protection includes rock, splash blocks, etc.)?
- ☐ Is the entrance flow path unimpeded by woody plants (trees, shrubs)?
- ☐ Is the underdrain at least 6 inches in diameter?
- ☐ Is the underdrain pipe made of accepted material (slotted PVC pipe conforming to ASTM C 3034 or equivalent HDPE pipe conforming to AASHTO 252M)?
- ☐ Does the slotted pipe have correct sizing and spacing of slots?
- ☐ Is the underdrain sloped at 0.5% or more?

- ☐ Are rigid observation pipes connected to underdrain every 250 to 300 feet of installed pipe?
- ☐ Do the observation pipe wells/clean outs extend 6 inches above top elevation of the planter box mulch and are they capped as required?
- ☐ Does the gravel underdrain bedding consist of the correct aggregate?
- ☐ Does the gravel underdrain bedding extend at least 6 inches below the underdrain and does it provide 1 foot depth around top and sides of pipe?
- ☐ If geotextile fabric is used in the underdrain design, does it meet minimum materials requirements?
- ☐ Is the underdrain elevated from the bottom of the planter box by 6 inches?
- ☐ Does the underdrain drain freely to the intended discharge point?
- ☐ Is an overflow device consisting of vertical PVC pipe included in design?
- ☐ Is the overflow riser at least 6 inches in diameter?
- ☐ Is the inlet to the riser 6 inches above planting soil and capped with a spider cap?
- ☐ Has a waterproof barrier consisting of a 30 mil geomembrane or equivalent been provided to protect foundations from moisture?
- ☐ Is planting soil composed of correct aggregate (60-70% sand; 30-40% compost) and gradation, and free of stones, stumps and roots?
- ☐ Does compost have acceptable characteristics (see planting/storage media)?
- ☐ Is planter box covered with well-aged mulch, free of seeds, weeds, grass clippings, bark, soil and roots, and at least 2-3 inches thick?
- ☐ Do all soil minerals meet requirements?
- ☐ Is all planter box vegetation tolerant of summer drought, ponding fluctuations, and saturated soil conditions for 48 to 72 hours?
- ☐ Have an adequate number of different plant species been incorporated into the planter box design (It is recommended that 3 tree, 3 shrub, and 3 herbaceous groundcover species be included)?
- ☐ Have native plants been used to the maximum extent practicable?
- ☐ Have only slow-release fertilizers been included in the design?
- ☐ Have arrangements been made to replace planter box mulch layer annually?

- ☐ Have low-maintenance plants been selected for design?
- ☐ Has an effort been made to ensure that no treated wood or galvanized metal is used anywhere within the planter box design?

BIO-3 Proprietary Biotreatment Device Checklist

- ☐ Has the proprietary biotreatment device been selected from the list provided in the manual of from another Ventura County- approved list?
- ☐ Has the vendor been contacted for the latest design guidance on cartridge selection?
- ☐ Has the proprietary biotreatment device been installed as directed by the vendor?
- ☐ Have appropriate maintenance and operation arrangements been made to ensure upkeep of the device?
- ☐ Has the biotreatment device been sized to capture and treat the water quality design flow?

BIO-4 Vegetated Swale Checklist

- ☐ Does the climate provide adequate conditions for maintaining a vegetative cover? Has adequate vegetation been chosen given the climate?
- ☐ Is the grade in the area shallow so as to not allow ponding?
- ☐ Is the swale compatible with existing flood control functions?
- ☐ Has the swale been designed with a depth of one foot or less?
- ☐ Is the overall depth from the top of the side walls to the bottom of the swale at least 12 inches?
- ☐ Is the swale bottom width at least 2 feet?
- ☐ Is the swale bottom width no greater than 10 feet, or 16 feet with a dividing berm?
- ☐ If the swale is required to convey flood flows in addition to the water quality design flow, has the swale been designed for the flood control design storm and does it include 2 feet of freeboard?
- ☐ Have gradual meandering bends been incorporated into the design?
- ☐ Is the longitudinal slope (in direction of flow) between 1% and 6%?
- ☐ Has an underdrain been provided if soils are poorly drained and longitudinal slope is less than 1.5%? Has a soils report been provided if this is the case?
- ☐ If the longitudinal slope is greater than 6%, have appropriate check dams with vertical drops of 12 inches or less been provided in the design to reduce the slope?
- ☐ Is the horizontal slope at the bottom of the swale flat to discourage channeling?
- ☐ Has the swale been designed so that the water depth does not exceed 4 inches or 2/3 the height of vegetation (2 inches in frequently mowed turf swales?
- ☐ Does the swale length provide a minimum hydraulic residence time of 7 minutes?
- ☐ If soil and slope conditions require it, has an acceptable low flow drain been installed?
- ☐ Has the swale been designed to convey the SQDF?

- ☐ Has the swale been sized as recommended in Chapter 6 (also see worksheet, Appendix E)?
- ☐ Has the swale been designed as a flow-through channel or has a high-flow bypass been incorporated into the design for flows higher than the water quality design flow?
- ☐ Has inflow been directed towards the upstream end of the swale or, at a minimum, evenly over the length of the swale?
- ☐ If the swale is online, has it been designed to convey flows up to the post-development 100 year 24 hour storm, with freeboard, and velocities below 3 ft/s?
- ☐ If the swale is off-line, has it been designed to convey the water quality design flow rate using a flow splitter with velocities below 1 ft/s?
- ☐ If check dams are incorporated in the design, have flow spreaders been added at the toe of each vertical drop?
- ☐ If curb cuts are used, has pavement been placed 1 – 2 inches above the elevation of the vegetated area?
- ☐ Is the swale inflow designed to function long term with minimal maintenance?
- ☐ Has flow spreading at the inlet of the swale been achieved by a leveled anchored flow spreader or similar method?
- ☐ Does the flow spreader project a minimum of 2 inches above the ground surface with appropriately spaced notches and extend horizontally beyond facility to prevent erosion
- ☐ If an underdrain is required, does it meet appropriate criteria (PVC or equivalent, correct slot spacing and sizing, 6 inches minimum in diameter, sloped at 0.5%)?
- ☐ Is there gravel bedding at least 6 inches below and 1 foot to the top and sides of the underdrain?
- ☐ If a geotextile is included in the design, does it meet requirements?
- ☐ Does gravel drainage layer meet recommended criteria?
- ☐ Does swale divider, if included, meet criteria (minimum height of 1 inch above flow, slopes no steeper than 2H:1V, stable foundation)?

- ☐ Has swale soil been amended with compost if organic content is less than 10%?
- ☐ Have appropriate, hardy and native plants been used to the maximum extent practical?
- ☐ Is vegetative cover at least 4 inches in height (ideally 6 inches)?
- ☐ Has the swale been located away from trees that may drop leaves or provide insufficient sunlight?

BIO-5 Vegetated Filter Strip Checklist

- ☐ Is the slope of the filter strip designed to avoid both erosive flows and ponding?
- ☐ Has the strip been designed to evenly distribute flow across width and promote sheet flow?
- ☐ Does the width of the filter strip extend across the full width of the tributary area?
- ☐ Is the upstream boundary of the filter located contiguous to developed area?
- ☐ If filter strip is used for water quality purposes, is the length between 15 and 150 feet (25 feet preferred)? If the strip is used for pretreatment, is it at least 4 feet in length?
- ☐ Is the slope of the strip parallel to the direction of flow between 2% and 6%?
- ☐ Is the lateral slope (perpendicular to flow) of the strip 4% or less?
- ☐ Is grading across strip even?
- ☐ Has the top of the strip been installed 2 to 5 inches below any adjacent pavement (a beveled transition is also acceptable)?
- ☐ Are the top and toe of the slope as flat as possible (graded flat for engineered filter strips) to encourage sheet flow and prevent erosion?
- ☐ Has the design flow been calculated using the SQDF (see worksheet)?
- ☐ Has the design flow depth been calculated using a modified Manning's equation (see worksheet)?
- ☐ Have the design velocity and length been calculated using the design flow and design flow depth as recommended (see worksheet)?
- ☐ Has a flow spreader been implemented to uniformly distribute contributing flow along width of filter strip?
- ☐ If a gravel flow spreader is used, is it at least 6 inches deep, 12 inches wide and a minimum or 1 inch below the paved surface?
- ☐ Has the gravel flow spreader been leveled even where ground is not level?
- ☐ If the gravel flow spreader is placed along a roadway, have LA county design specifications been consulted and implemented?

- ☐ If a notched curb spreader and through-curb spreader are used, have they been used in conjunction with a gravel spreader?
- ☐ Have curb port/interrupted curb openings been spaced at intervals of at least every 6 feet?
- ☐ Do the curb port/interrupted curb openings have a width of at least 11 inches?
- ☐ Does 15% or more of the curb length consist of open ports and does each port discharge no more than 10% of the flow?
- ☐ Have energy dissipaters (such as a riprap pad) been used if a sudden slope drop occurs?
- ☐ Has access been provided at the upper edge of filter strip for mowing equipment and to enable maintenance of spreader?
- ☐ Is the design water depth 1 inch or less?
- ☐ Does the design velocity not exceed 1 foot per second?
- ☐ If the organic content of the filter strip soil does not exceed 10%, has the soil been amended with at least 2 inches of well-rotted acceptable compost at a depth of 6 inches?
- ☐ Is filter strip uniformly graded and densely vegetated with erosion-resistant grasses (preferably native or adapted species)?
- ☐ Has irrigation been provided to establish grasses?
- ☐ Have maintenance arrangements been made to maintain grass at a height of 2 to 4 inches?
- ☐ Have trees and shrubs been limited along the filter strip?
- ☐ Has an effort been made to ensure that no treated wood or galvanized metal is used anywhere within the design?

BIO-6 Green Roof Checklist

- ☐ Is the roof shallow enough to support a green roof (<25% slope)?
- ☐ Are the roof supports sufficient to support additional weight of soil, water, vegetation, and a drainage layer (if needed) [a licensed structural engineer should be consulted]?
- ☐ Has an appropriate waterproof membrane been placed below the green roof?
- ☐ Has an appropriate drainage layer been incorporated in the design (if required)?
- ☐ Has an appropriate soil mix been used in the design to allow for drainage, support vegetative growth, and that is not excessively heavy when wet?
- ☐ Has vegetation been carefully selected to improve aesthetics, resist erosion, withstand extreme environments, and tolerate drought without the need for fertilizers and pesticides and without a lot of maintenance requirements (see Appendix H for a recommended plant list)?
- ☐ Have native plants been chosen to the maximum extent practical?
- ☐ If trees or shrubs are incorporated, has an adequate soil depth been provided and is the additional soil depth supported by the roof structure?
- ☐ Has irrigation been provided to establish vegetation?
- ☐ Does vegetation cover 90% of the total area?
- ☐ Is the green roof located in an area without excessive shade to avoid poor vegetative growth?
- ☐ Is there an appropriate drain pipe or gutter to convey any runoff from roof to a stormwater BMP or stormwater conveyance system?

FILT-1 Sand Filter Checklist

☐ Has sand filter been located away from trees and areas that could contribute eroded sediment?

☐ If there is a chance for sediment to be present in flow to be treated, has pretreatment been provided?

☐ Does site have adequate relief to permit vertical percolation through sand filter and into conveyance system?

☐ Has pretreatment (vegetated swale or filter strip, hydrodynamic separator) been adequately provided to reduce the sediment load entering the filter?

☐ Has the sand filter been sized to capture the SQDV?

☐ Has the sand filter been designed with a 1.5:1 length to width ratio or greater?

☐ Is the filter bed depth at least 2 feet (3 feet preferred)?

☐ Is the depth of water storage over the filter bed 6 feet or less?

☐ Is the overflow structure designed to pass the water quality design storm?

☐ Has the sizing of the filter been determined using the adapted Darcy's Law equation recommended in the sizing methodology section in Chapter 6 (also see worksheet, Appendix E)?

☐ Does the sand meet the recommended specifications (0.2-0.35 mm diameter, Cu < 3, ASTM C 33 size gradation, etc.)?

☐ Has an underdrain been employed in the design? [Examples: central underdrain w/lateral pipes, longitudinal pipes, single pipe for small filters]

☐ Is the underdrain placed in an 8 inch minimum gravel backfill or drain rock bed?

☐ Are all underdrain pipes and connectors 6 inches or greater with clean-out risers of equal diameter?

☐ Have clean-out risers been placed at the terminal ends of all pipes and extend to the surface of the filter?

☐ Has a valve box been provided for access to the clean-outs and is it water tight?

☐ Are underdrain pipes laid with perforations downward, and are perforations at least ½ inch in diameter?

- ☐ Are all lateral collection pipes within 9 feet or less of each other (perpendicular distance)?
- ☐ Have all pipes been placed with a minimum slope of 0.5%?
- ☐ Is the invert of the underdrain outlet above the seasonal high groundwater level?
- ☐ Is gravel backfill present around the underdrain pipe at least 6 inches below and to the sides of the pipe and 8 inches above the pipe?
- ☐ Does the bottom gravel have a diameter of at least 2 times the size of the perforated openings to the drainage system and meet other specifications (specific gravity of 2.5 or more, rounded, free of debris)?
- ☐ Has an appropriate geotextile layer (see underdrain section) or 2-inch transition layer been placed between the sand layer and the drain rock/ gravel backfill layer?
- ☐ Has a flow spreader been installed at the inlet along one side of the filter (long side of the filter if L: W is 2:1 or greater; 20% of perimeter for curved or irregular shape)?
- ☐ Has erosion protection been provided along the first foot of the sand bed adjacent to the flow spreader (i.e. geotextile weighted with sand bags; quarry spalls)?
- ☐ Has no topsoil, clay, or sod (except sod grown in sand) has been added to the sand filter bed?
- ☐ Has vegetation been selected properly (i.e. must withstand drought, heavy saturation, etc.)?
- ☐ Are no permanent structures built on top of the sand filter bed?
- ☐ No large shrubs or trees should be planted in sand filter bed or within 15 feet of inlet or outlet pipes
- ☐ Have native plants been used to the maximum extent practicable?
- ☐ Has an emergency overflow structure been provided?
- ☐ Are interior side slopes above water quality design depth no steeper than 3:1 H:V?
- ☐ Are exterior side slopes no steeper than 2:1 H:V?
- ☐ If pond walls are vertical retaining walls, do they meet recommended specifications (see side slopes section)?

☐ Do embankments meet appropriate criteria [top width or 20 feet, constructed on native consolidated soil, in accordance with standard specifications, proper excavation, constructed of appropriate compacted soil]?

☐ Are maintenance access roads/ramps to filter provided?

☐ Have trees and shrubs been planted further than 10 feet away from inlet and outlet pipes (50 feet for ‘water-seeking’ plants such as willows and poplars)?

☐ Have prohibited non-native plants been removed from the site?

☐ Has an effort been made to ensure that no treated wood or galvanized metal is used anywhere within the planter box design?

FILT-2 Cartridge Media Filter

- ☐ Has the vendor been contacted for the latest design guidance on cartridge selection?
- ☐ Has the cartridge media filter been provided with a system to completely drain the system and prevent vector annoyances?
- ☐ Has the cartridge media filter been sized to capture and treat the SQDF?
- ☐ Have site considerations been taken into account when sizing the cartridge media filter and selecting features (often vendor websites offer assistance with this)?

INF-1 Infiltration Trench Checklist

- ☐ Has the infiltration trench been located away from steep slopes (>25%)?
- ☐ Is the infiltration trench set back from structures and leach fields?
- ☐ Is there at least 10 feet or vertical separation between the bottom of the infiltration trench and the shallow groundwater table?
- ☐ Is the depth to bedrock adequate to provide proper infiltration?
- ☐ Has the site been checked to ensure that no preexisting contamination is present?
- ☐ Does the site have low sediment loading rates to prevent infiltration trench clogging?
- ☐ Has a soil assessment report been completed, which determines the suitability of the site for an infiltration trench, recommends a design infiltration rate, identifies the high depth to groundwater table surface elevation, and examines how the stormwater runoff will move in the soil?
- ☐ Has a geotechnical investigation and report been provided if needed?
- ☐ Has the infiltration trench been located at a site that does not receive run off from sites that store or use chemicals or hazardous waste outside?
- ☐ Has the infiltration trench been set back from existing septic system drain fields and drinking water wells?
- ☐ Has pretreatment been provided with a vegetated swale, filter strip, sand filter or proprietary device?
- ☐ Is the trench at least 2 feet wide and 3 to 5 feet deep?
- ☐ Is the longitudinal slope of the trench 3% or less?
- ☐ Is the top layer of the media filter gravel/choking stone/geotextile fabric if flow is sheet flow and 12 inches of surface soil if flow enters through an underground pipe?
- ☐ Is middle layer of media filter 3-5 feet of washed 1.5 to 3 in. gravel with void space of 30 to 40%?
- ☐ Is bottom layer of media filter 6” of clean, washed sand?
- ☐ Have one or more observation wells been installed?

- ☐ Do observation wells consist of recommended slotted 4-6 inch diameter PVC well screen capped with lockable, above-ground lid?
- ☐ Has the infiltration trench been sized to capture and infiltrate the SUSMP defined water quality design volume?
- ☐ Has the infiltration trench been designed to infiltrate all runoff within 72 hours?
- ☐ Has the maximum depth of runoff, ponding depth/trench depth and infiltrating surface area been calculated using recommended design equations (see sizing methodology section/worksheet)?
- ☐ Is the bottom of the infiltration bed native soil, over-excavated to at least one foot in depth and replaced uniformly (with 2-4 inches of coarse sand amendments) without compaction?
- ☐ Has all vertical piping been classified correctly (see drainage section in manual)?
- ☐ Has an observation well been incorporated into the design to ensure that the 72 hour maximum drawdown time is met?
- ☐ Has an overflow route been provided to safely convey flows that overtop the facility or in the case that the facility becomes clogged?
- ☐ Has the overflow channel been designed to safely convey flows from peak design storm to a downstream conveyance system or acceptable discharge point?
- ☐ Has the infiltration trench been kept free of vegetation, and is all existing vegetation surrounding the trench been planted away from trench to avoid drip lines overhanging the facility?
- ☐ Is there safe maintenance access provided to the site for both wet and dry conditions?
- ☐ Has an access road along the length of the trench been provided if there is no existing road or parking lot that can be used for maintenance access?
- ☐ Has access to “operate a backhoe at ‘arms length’” been provided?
- ☐ Was the entire area draining to the facility stabilized before construction began?
- ☐ Have you ensured that the infiltration trench is not hydraulically connected to the storm water conveyance system?

- ☐ If heavy construction material was used to compact subgrade (not recommended), has the infiltrative capacity of the soil been restored via tilling or aerating prior to placing the infiltration bed?
- ☐ Were the exposed subgrade soils inspected by a civil engineer prior to construction to confirm suitable soil conditions for the infiltration facility?

INF-2 Drywell Checklist

- ☐ Has the drywell been located away from steep slopes (>25%)?
- ☐ Is the drywell set back from structures and leach fields?
- ☐ Is there at least 10 feet or vertical separation between the bottom of the drywell and the shallow groundwater table?
- ☐ Is the depth to bedrock adequate to provide proper infiltration?
- ☐ Has the site been checked to ensure that no preexisting contamination is present?
- ☐ Does the site have low sediment loading rates to prevent drywell from clogging?
- ☐ Has pretreatment been provided for all non-rooftop runoff flowing to the drywell?
- ☐ Has a geotechnical investigation and report been provided to ensure site meets specifications for an infiltration facility (including soil infiltration rate, groundwater separation, and no steep slopes)?
- ☐ Has a soil assessment report been completed, which determines the suitability of the site for an drywell, recommends a design infiltration rate, identifies the high depth to groundwater table surface elevation, and examines how the stormwater runoff will move in the soil?
- ☐ Has the drywell been located at a site that does not receive run off from sites that store or use chemicals or hazardous waste outside?
- ☐ Has the drywell been set back from existing septic system drain fields and drinking water wells?
- ☐ Has pretreatment been provided to prevent sediment and other large particulates?
- ☐ Is the surface area of the drywell large enough to infiltrate the storage volume in 72 hours based on maximum allowable depth?
- ☐ Is the top layer of the media filter gravel/choking stone/geotextile fabric if flow is sheet flow and 12 inches of surface soil if flow enters through an underground pipe (pipe should be fitted with a screen)?
- ☐ Is middle layer of media filter 3-5 feet of washed 1.5 to 3 in. gravel with void space of 30 to 40%?
- ☐ Is bottom layer of media filter 6” of clean, washed sand?

- ☐ Have one or more observation wells been installed?
- ☐ Do observation wells consist of recommended slotted 4-6 inch diameter PVC well screen capped with lockable, above-ground lid?
- ☐ Has the drywell been sized to capture and infiltrate the SUSMP defined water quality design volume?
- ☐ Has the drywell been designed to infiltrate all runoff within 72 hours?
- ☐ Has a long term percolation rate of 10% of the measured percolation rate been used in design (due to occlusion and particulate accumulation)?
- ☐ Has the maximum depth of runoff, ponding depth/trench depth and infiltrating surface area been calculated using recommended design equations (see sizing methodology section/worksheet)?
- ☐ Is the bottom of the infiltration bed native soil, over-excavated to at least one foot in depth and replaced uniformly (with 2-4 inches of coarse sand amendments) without compaction?
- ☐ Has all vertical piping been classified correctly (see drainage section in manual)?
- ☐ Has an observation well been incorporated to ensure that the 72 hour maximum drawdown time is met?
- ☐ Has an overflow route been provided to safely convey flows that overtop the facility or in the case that the facility becomes clogged?
- ☐ Has the overflow channel been designed to safely convey flows from peak design storm to a downstream conveyance system or acceptable discharge point?
- ☐ Has the drywell been kept free of vegetation, and is all existing vegetation surrounding the trench been planted away from trench to avoid drip lines overhanging the facility?
- ☐ Is there safe maintenance access provided to the site for both wet and dry conditions?
- ☐ Has maintenance access been provided?
- ☐ Was the entire area draining to the facility stabilized before construction began?
- ☐ Have you ensured that the infiltration trench is not hydraulically connected to the storm water system?

- ☐ If heavy construction material was used to compact subgrade (not recommended), has the infiltrative capacity of the soil been restored via tilling or aerating prior to placing the infiltration bed?
- ☐ Were the exposed subgrade soils inspected by a civil engineer prior to construction to confirm suitable soil conditions for the infiltration facility?

INF-3 Proprietary Infiltration BMPs Checklist

- ☐ Has the infiltration facility been located away from steep slopes (>25%)?
- ☐ Is the infiltration facility set back from structures and leach fields?
- ☐ Is there at least 10 feet or vertical separation between the bottom of the infiltration facility and the shallow groundwater table?
- ☐ Is the depth to bedrock adequate to provide proper infiltration?
- ☐ Has the site been checked to ensure that no preexisting contamination is present?
- ☐ Does the site have low sediment loading rates to prevent infiltration facility clogging?
- ☐ Has pretreatment been provided to prevent premature failure (If infiltration facility fails, complete construction is required)?
- ☐ Has infiltration facility been designed to receive runoff only from sections of the site that have been stabilized?
- ☐ If infiltration facility fails, complete construction is required
- ☐ Has a geotechnical investigation and report been provided to ensure site meets specifications for an infiltration facility (including soil infiltration rate, groundwater separation, and no steep slopes)?
- ☐ Has a soil assessment report been completed, which determines the suitability of the site for an infiltration trench, recommends a design infiltration rate, identifies the high depth to groundwater table surface elevation, and examines how the stormwater runoff will move in the soil?
- ☐ Has the infiltration trench been located at a site that does not receive run off from sites that store or use chemicals or hazardous waste outside?
- ☐ Has the infiltration BMP been sized to capture and treat the water quality design volume?
- ☐ Has a long term percolation rate of 10% of the measured percolation rate been used in design (due to occlusion and particulate accumulation)?
- ☐ Have the recommended sizing guidelines set by the vendor been referenced and used for selection and use of infiltration facility?

INF-4 Permeable Pavement Checklist

- ☐ Has the permeable pavement been located away from steep slopes (>25%)?
- ☐ Is the permeable pavement set back from structures and leach fields?
- ☐ Is there at least 10 feet or vertical separation between the bottom of the permeable pavement and the shallow groundwater table?
- ☐ Is the depth to bedrock adequate to provide proper infiltration?
- ☐ Has the site been checked to ensure that no preexisting contamination is present?
- ☐ Does the site have low sediment loading rates to prevent infiltration trench clogging?
- ☐ Has the permeable pavement been designed to receive runoff only from sections of the site that have been stabilized?
- ☐ Has a geotechnical investigation and report been provided to ensure site meets specifications for an infiltration facility (including soil infiltration rate, groundwater separation, and no steep slopes)?
- ☐ Has a soil assessment report been completed, which determines the suitability of the site for an infiltration trench, recommends a design infiltration rate, identifies the high depth to groundwater table surface elevation, and examines how the stormwater runoff will move in the soil?
- ☐ Has the permeable pavement been located at a site that does not receive run off from sites that store or use chemicals or hazardous waste outside?
- ☐ Has the run off been assessed for necessity of pretreatment?
- ☐ If pretreatment is required, has it been provided to treat run on before it reaches permeable pavement?
- ☐ Has the infiltration BMP been sized to capture and treat the water quality design volume?
- ☐ Have the infiltration capabilities of the site been assessed (i.e. full, partial, or no infiltration allowed)?
- ☐ If no infiltration is allowed, has an underdrain been prohibited?

- ☐ If permeable pavement is located on a site with a slope greater than 2%, has the area been terraced to prevent lateral flow through subsurface?
- ☐ Has the permeable pavement been designed to infiltrate flows through four different layers (incl. top wearing layer, stone reservoir, and transition layers) of material (or through a similar system)?
- ☐ Has the depth of each layer (and void space), along with the hydrology, hydraulics, and structural requirements of the site been determined and approved by a licensed civil engineer?
- ☐ If proprietary permeable pavement is used (i.e. concrete or other pavers), have the design requirements and installation steps been obtained from the vendor and referenced in the selection and construction of the permeable pavement?
- ☐ Has the permeable pavement been designed to drain in less than 72 hours and allowed to dry out periodically?
- ☐ Has a long term percolation rate of 10% of the measured percolation rate been used in design (due to occlusion and particulate accumulation)?
- ☐ Has an overflow mechanism been included in the pavement design?
- ☐ If the overflow mechanism employed is perimeter control, have controls such as a perimeter vegetated swale, perimeter Bioretention, storm drain inlets, or other acceptable control been implemented?
- ☐ If the overflow mechanism employed are overflow pipes, have the pipes been connected to the underdrain, are they located away from vehicular traffic, and is the top of the pipe fitted with a screen?
- ☐ Has the pavement been laid close to level with bottom of base layers level to ensure uniform infiltration?
- ☐ Are site materials stored away from permeable pavement?
- ☐ Has landscaping and stabilization of adjacent areas been completed before installation of pavement?

GS-1 Hydrodynamic Separation Device Checklist

- ☐ Has the vendor been contacted for the latest model and design guidance prior to selection of device?
- ☐ Has the device been sized to capture and treat the water quality design flow rate?
- ☐ Has the vendor been contacted for sizing and installation guidance?
- ☐ Has periodic maintenance been scheduled and budgeted for?

GS-2 Catch Basin Insert Checklist

- ☐ Has the vendor been contacted for the latest model and design guidance prior to selection of device?
- ☐ Has the insert been sized to capture and treat the water quality design flow rate?
- ☐ Has the vendor been contacted for sizing and installation guidance?
- ☐ Has periodic maintenance been scheduled and budgeted for?

APPENDIX H : STORMWATER CONTROL MEASURE ACCESS AND MAINTENANCE AGREEMENTS

(Long Form)

Recorded at the request of:

City of ____

After recording, return to:

City of ____

City Clerk

Stormwater Treatment Device Access and Maintenance Agreement

OWNER:

PROPERTY ADDRESS:____

APN:

THIS AGREEMENT is made and entered into in_____________________, California, this___day of__________________, by and between__________________ ______________________, hereinafter referred to as "Owner" and the CITY OF____ ____________________, a municipal corporation, located in the County of Ventura, State of California hereinafter referred to as "CITY";

WHEREAS, the Owner owns real property ("Property") in the City of __________, County of Ventura, State of California, more specifically described in Exhibit "A" and depicted in Exhibit "B", each of which exhibits is attached hereto and incorporated herein by this reference;

WHEREAS, at the time of initial approval of development project known as____ ____within the Property described herein, the City required the project to employ on-site control measures to minimize pollutants in urban runoff;

WHEREAS, the Owner has chosen to install a ______________________________ ____, hereinafter referred to as "Device", as the on-site control measure to minimize pollutants in urban runoff;

WHEREAS, said Device has been installed in accordance with plans and specifications accepted by the City;

WHEREAS, said Device, with installation on private property and draining only private property, is a private facility with all maintenance or replacement, therefore, the sole responsibility of the Owner in accordance with the terms of this Agreement;

WHEREAS, the Owner is aware that periodic and continuous maintenance, including, but not necessarily limited to, filter material replacement and sediment removal, is required to assure peak performance of Device and that, furthermore, such maintenance activity will require compliance with all Local, State, or Federal laws and regulations, including those pertaining to confined space and waste disposal methods, in effect at the time such maintenance occurs;

NOW THEREFORE, it is mutually stipulated and agreed as follows:

1) Owner hereby provides the City of City's designee complete access, of any duration, to the Device and its immediate vicinity at any time, upon reasonable notice, or in the event of emergency, as determined by City's Director of Public Works no advance notice, for the purpose of inspection, sampling, testing of the Device, and in case of emergency, to undertake all necessary repairs or other preventative measures at owner's expense as provided in paragraph 3 below. City shall make every effort at all times to minimize or avoid interference with Owner's use of the Property.

2) Owner shall use its best efforts diligently to maintain the Device in a manner assuring peak performance at all times. All reasonable precautions shall be exercised by Owner and Owner's representative or contractor in the removal and extraction of material(s) from the Device and the ultimate disposal of the material(s) in a manner consistent with all relevant laws and regulations in effect at the time. As may be requested from time to time by the City, the Owner shall provide the City with documentation identifying the material(s) removed, the quantity, and disposal destination.

3) In the event Owner, or its successors or assigns, fails to accomplish the necessary maintenance contemplated by this Agreement, within five (5) days of being given written notice by the City, the City is hereby authorized to cause any maintenance necessary to be done and charge the entire cost and expense to the Owner or Owner's successors or assigns, including administrative costs, attorneys fees and interest thereon at the maximum rate authorized by the Civil Code from the date of the notice of expense until paid in full.

4) The City may require the owner to post security in form and for a time period satisfactory to the city of guarantee of the performance of the obligations stated herein. Should the Owner fail to perform the obligations under the Agreement, the City may, in the case of a cash bond, act for the Owner using the proceeds from it, or in the case of a surety bond, require the sureties to perform the obligations of the Agreement. As an additional remedy, the Director may withdraw any previous stormwater related approval with respect to the

property on which a Device has been installed until such time as Owner repays to City it's reasonable costs incurred in accordance with paragraph 3 above.

5) This agreement shall be recorded in the Office of the Recorder of Ventura County, California, at the expense of the Owner and shall constitute notice to all successors and assigns of the title to said Property of the obligation herein set forth, and also a lien in such amount as will fully reimburse the City, including interest as herein above set forth, subject to foreclosure in event of default in payment.

6) In event of legal action occasioned by any default or action of the Owner, or its successors or assigns, then the Owner and its successors or assigns agree(s) to pay all costs incurred by the City in enforcing the terms of this Agreement, including reasonable attorney's fees and costs, and that the same shall become a part of the lien against said Property.

7) It is the intent of the parties hereto that burdens and benefits herein undertaken shall constitute covenants that run with said Property and constitute a lien there against.

8) The obligations herein undertaken shall be binding upon the heirs, successors, executors, administrators and assigns of the parties hereto. The term "Owner" shall include not only the present Owner, but also its heirs, successors, executors, administrators, and assigns. Owner shall notify any successor to title of all or part of the Property about the existence of this Agreement. Owner shall provide such notice prior to such successor obtaining an interest in all or part of the Property. Owner shall provide a copy of such notice to the City at the same time such notice is provided to the successor.

9) Time is of the essence in the performance of this Agreement.

10) Any notice to a party required or called for in this Agreement shall be served in person, or by deposit in the U.S. Mail, first class postage prepaid, to the address set forth below. Notice(s) shall be deemed effective upon receipt, or seventy-two (72) hours after deposit in the U.S. Mail, whichever is earlier. A party may change a notice address only by providing written notice thereof to the other party.

IF TO CITY:	IF TO OWNER:
______________________________	______________________________
______________________________	______________________________
______________________________	______________________________

IN WITNESS THEREOF, the parties hereto have affixed their signatures as of the date first written above.

APPROVED AS TO FORM:

____

City Attorney

OWNER:

____

Owner

Name: ______________________________

Title: ______________________________

CITY OF____________________:

____

Name:________________________

Title:________________________

OWNER:

____

Name:________________________________

Title:________________________________

ATTEST:

____

City Clerk Date

Notaries on Following Page

EXHIBIT A

(Legal Description)

EXHIBIT B

(Map/illustration)

(Short Form)

Recorded at the request of and mail to:

____

____

____

Covenant and Agreement Regarding

Stormwater Treatment Device Maintenance

The undersigned hereby certify that we are the owners of hereinafter legally described real property located in the City of _________________________, County of Ventura, State of California.

Legal Description: ____

____

as recorded in Book _____________________, Page________,Records of Ventura County,

which property is located and known as **(Address):** ______________________________

____.

And in consideration of the City of ____________ allowing__________________________

____

on said property, we do hereby covenant and agree to and with said City to maintain according to the Maintenance Plan (Attachment 1), all structural stormwater treatment devices including the following:

____

____.

This Covenant and Agreement shall run all of the above described land and shall be binding upon ourselves, and future owners, encumbrances, their successors, heirs, or assignees and shall continue in effect until released by the authority of the City upon submittal of request, applicable fees, and evidence that this Covenant and Agreement is no longer required by law.

NOTARIES ON FOLLOWING PAGE

APPENDIX I : STORMWATER CONTROL MEASURE MAINTENANCE PLAN GUIDELINES AND CHECKLISTS

Included in this appendix are a series of checklists that can be used by both inspectors and maintenance personnel to ensure that observed deficiencies in BMPs are maintained appropriately. The BMP Inspection/Maintenance Checklists are presented in the following order:

1) Bioretention/Planter Box

25) Vegetated Swale Filter

26) Vegetated Filter Strip

27) Sand Filter

28) Infiltration BMPs

29) Permeable Pavement

30) Constructed Treatment Wetland

31) Wet Retention Basin

32) Dry Extended Detention Basin

33) Proprietary Devices

I.1 Bioretention/Planter Box Inspection and Maintenance Checklist

Date: ______________________ Work Order # __________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ______________________ Inspector(s): __________________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0, 1, or 2)†	Date Maintenance Performed	Comments or Action(s) Taken to Resolve Issue
Appearance	Untidy			
Trash and Debris Accumulation	Trash, plant litter and dead leaves accumulated on surface.			
Vegetation	Unhealthy plants and appearance.			
Irrigation	Functioning incorrectly (if applicable).			
Inlet	Inlet pipe blocked or impeded.			
Splash Blocks	Blocks or pads correctly positioned to prevent erosion.			
Overflow	Overflow pipe blocked or broken.			
Filter media	Infiltration design rate is met (e.g., drains 36-48 hours after moderate - large storm event).			

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

I.2 Vegetated Swale Filter Inspection and Maintenance Checklist

Date: ___________________________ Work Order # ____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: _________________________ Inspector(s): _____________________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0, 1, or 2)†	Date Maintenance Performed	Comments or Action(s) Taken to Resolve Issue
Appearance	Untidy			
Trash and Debris Accumulation	Trash and debris accumulated in the swale.			
Vegetation	When the grass becomes excessively tall (greater than 10-inches); when nuisance weeds and other vegetation start to take over.			
Excessive Shading	Vegetation growth is poor because sunlight does not reach swale. Evaluate vegetation suitability.			
Poor Vegetation Coverage	When vegetation is sparse or bare or eroded patches occur in more than 10% of the swale bottom. Evaluate vegetation suitability.			
Sediment Accumulation	Sediment depth exceeds 2 inches or covers more than 10% of design area.			
Standing Water	When water stands in the swale between storms and does not drain freely.			
Flow spreader or Check Dams	Flow spreader or check dams uneven or clogged so that flows are not uniformly distributed through entire swale width.			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0, 1, or 2)†	Date Maintenance Performed	Comments or Action(s) Taken to Resolve Issue
Constant Baseflow	When small quantities of water continually flow through the swale, even when it has been dry for weeks and an eroded, muddy channel has formed in the swale bottom.			
Inlet/Outlet	Inlet/outlet areas clogged with sediment and/or debris.			
Erosion/ Scouring	Eroded or scoured swale bottom due to flow channelization, or higher flows. Eroded or rilled side slopes.			
	Eroded or undercut inlet/outlet structures			

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

I.3 Vegetated Filter Strip Inspection and Maintenance Checklist

Date: ____________________________ Work Order # ____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: __________________________ Inspector(s): ____________________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0, 1 or 2)†	Date Maintenance Performed	Comments or Action(s) Taken to Resolve Issue
Appearance	Untidy			
Trash and Debris Accumulation	Trash and debris accumulated on the filter strip.			
Vegetation	When the grass becomes excessively tall (greater than 10-inches); when nuisance weeds and other vegetation starts to take over.			
Excessive Shading	Grass growth is poor because sunlight does not reach swale. Evaluate grass species suitability.			
Poor Vegetation Coverage	When grass is sparse or bare or eroded patches occur in more than 10% of the swale bottom. Evaluate grass species suitability.			
Erosion/Scouring	Eroded or scoured areas due to flow channelization, or higher flows.			
Sediment Accumulation on Grass	Sediment depth exceeds 2 inches.			
Flow spreader	Flow spreader uneven or clogged so that flows are not uniformly distributed through entire filter width.			

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

I.4 Sand Filter Inspection and Maintenance Checklist

Date: ___________________________ Work Order # ____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: _________________________ Inspector(s): _____________________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Trash & Debris	Any trash and debris which exceed 5 cubic feet per 1,000 square feet of filter bed area (one standard garbage can). In general, there shall be no visual evidence of dumping. If less than threshold all trash and debris will be removed as part of next scheduled maintenance.			
Inlet erosion	Visible evident of erosion occurring near flow spreader outlets.			
Slow drain time	Standing water long after storm has passed (after 24 to 48 hours) and/or flow through the overflow pipes occurs frequently.			
Concentrated Flow	Flow spreader uneven or clogged so that flows are not uniformly distributed across the sand filter.			
Appearance of poisonous, noxious or nuisance vegetation	Excessive grass and weed growth. Noxious weeds, woody vegetation establishing, Turf growing over rock filter			
Standing Water	Standing water long after storm has passed (after 24 to 48 hours), and/or flow through the overflow pipes occurs frequently.			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Tear in Filter Fabric	When there is a visible tear or rip in the filter fabric allowing water to bypass the fabric.			
Pipe Settlement	If piping has visibly settled more than 1 inch.			
Filter Media	Drawdown of water through the media takes longer than 1 hour and/or overflow occurs frequently.			
Short Circuiting	Flows do not properly enter filter cartridges.			

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

I.5 Infiltration BMP Inspection and Maintenance Checklist

Date: ____________________ Work Order # ____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ____________________ Inspector(s): ____________________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) Taken to Resolve Issue
Appearance, vegetative health	Mowing and trimming vegetation is needed to prevent establishment of woody vegetation, and for aesthetic and vector reasons.			
Vegetation	Poisonous or nuisance vegetation or noxious weeds.			
	Excessive loss of turf or ground cover (if applicable).			
Trash & Debris	Trash and debris > 5 cf/1,000 sf (one standard size garbage can).			
Contaminants and Pollution	Any evidence of oil, gasoline, contaminants or other pollutants.			
Erosion	Undercut or eroded areas at inlet or outlet structures.			
Sediment and Debris	Accumulation of sediment, debris, and oil/grease on surface, inflow, outlet or overflow structures.			
Sediment and Debris	Accumulation of sediment and debris, in sediment forebay and pretreatment devices.			
Water drainage rate	Standing water, or by visual inspection of wells (if available), indicates design drain times are not being achieved (i.e., within 72 hours).			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) Taken to Resolve Issue
Media clogging surface layer	Lift surface layer (and filter fabric if installed) and check for media clogging with sediment (function may be able to be restored by replacing surface aggregate/filter cloth).			
Media clogging	Lift surface layer (and filter fabric if installed) and check for media clogging with sediment (partial or complete clogging which may require full replacement).			

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

I.6 Permeable Pavement Inspection and Maintenance Checklist

Date: ______________________ Work Order # ________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ____________________ Inspector(s): _________________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Sediment Accumulation	Sediment is visible			
Missing gravel/sand fill	There are noticeable gaps in between pavers			
Weeds/mosses filling voids	Vegetation is growing in/on permeable pavement			
Trash and Debris Accumulation	Trash and debris accumulated on the permeable pavement.			
Dead or dying vegetation in adjacent landscaping	Vegetation is dead or dying leaving bare soil prone to erosion			
Surface clog	Clogging is evidenced by ponding on the surface			
Overflow clog	Excessive build up of water accompanied by observation of low flow in observation well (connected to underdrain system) If a surface overflow system is used, observation of an obvious clog			
Visual contaminants and pollution	Any visual evidence of oil, gasoline, contaminants or other pollutants.			
Erosion	Tributary area Exhibits signs of erosion Noticeably not completely stabilized			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Deterioration/ Roughening	Integrity of pavement is compromised (i.e., cracks, depressions, crumbling, etc.)			
Subsurface Clog	Clogging is evidenced by ponding on the surface and is not remedied by addressing surface clogging.			
†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.				

I.7 Constructed Treatment Wetland Inspection and Maintenance Checklist

Date: ______________________ Work Order # ________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ______________________ Inspector(s): ________________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Trash & Debris	Any trash and debris which exceed 5 cubic feet per 1,000 sf of basin area (one standard garbage can). In general, there shall be no visual evidence of dumping. If less than threshold all trash and debris will be removed as part of next scheduled maintenance. If trash and debris is observed blocking or partially blocking an outlet structure or inhibiting flows between cells, it shall be removed quickly			
Sediment Accumulation	Sediment accumulation in basin bottom that exceeds the depth of sediment zone plus 6 inches in the sediment forebay. If sediment is blocking an inlet or outlet, it shall be removed.			
Erosion	Erosion of basin's side slopes and/or scouring of basin bottom.			
Oil Sheen on Water	Prevalent and visible oil sheen.			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Noxious Pests	Visual observations or receipt of complaints of numbers of pests that would not be naturally occurring and could pose a threat to human or aquatic health.			
Water Level	First cell empty, doesn't hold water.			
Aesthetics	Minor vegetation removal and thinning. Mowing berms and surroundings			
Noxious Weeds	Any evidence of noxious weeds.			
Tree Growth	Tree growth does not allow maintenance access or interferes with maintenance activity (i.e., slope mowing, silt removal, vactoring, or equipment movements). If trees are not interfering, do not remove. Dead, diseased, or dying trees shall be removed.			
Settling of Berm	If settlement is apparent. Settling can be an indication of more severe problems with the berm or outlet works. A geotechnical engineer shall be consulted to determine the source of the settlement if the dike/berm is serving as a dam.			
Piping through Berm	Discernable water flow through basin berm. Ongoing erosion with potential for erosion to continue. A licensed geotechnical engineer shall be called in to inspect and evaluate condition and recommend repair of condition.			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Tree and Large Shrub Growth on Downstream Slope of Embankments	Tree and large shrub growth on downstream slopes of embankments may prevent inspection and provide habitat for burrowing rodents.			
Erosion on Spillway	Rock is missing and soil is exposed at top of spillway or outside slope.			
Gate/Fence Damage	Damage to gate/fence, including missing locks and hinges			
†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.				

I.8 Wet Retention Basin Inspection and Maintenance Checklist

Date: ___________________________ Work Order # ____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: _________________________ Inspector(s): _____________________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Trash & Debris	Any trash and debris which exceed 5 cubic feet per 1,000 sf of basin area (one standard garbage can) or if trash and debris is excessively clogging the outlet structure. If less than threshold all trash and debris will be removed as part of next scheduled maintenance.			
Sediment Accumulation	Sediment accumulation in basin bottom that exceeds the depth of the design sediment zone plus 6 inches, usually in the first cell.			
Erosion	Erosion of basin's side slopes and/or scouring of basin bottom.			
Oil Sheen on Water	Prevalent and visible oil sheen.			
Noxious Pests	Visual observations or receipt of complaints of numbers of pests that would not be naturally occurring and could pose a threat to human or aquatic health.			
Water Level	First cell empty, doesn't hold water.			
Algae Mats	Algae mats over more than 20% of the water surface.			
Aesthetics	Minor vegetation removal and thinning. Mowing berms and surroundings			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Noxious Weeds	Any evidence of noxious weeds.			
Tree Growth	Tree growth does not allow maintenance access or interferes with maintenance activity (i.e., slope mowing, silt removal, vactoring, or equipment movements). If trees are not interfering, do not remove. Dead, diseased, or dying trees shall be removed.			
Settling of Berm	If settlement is apparent. Settling can be an indication of more severe problems with the berm or outlet works. A geotechnical engineer shall be consulted to determine the source of the settlement if the dike/berm is serving as a dam.			
Piping through Berm	Discernable water flow through basin berm. Ongoing erosion with potential for erosion to continue. A licensed geotechnical engineer shall be called in to inspect and evaluate condition and recommend repair of condition.			
Tree and Large Shrub Growth on Downstream Slope of Embankments	Tree and large shrub growth on downstream slopes of embankments may prevent inspection and provide habitat for burrowing rodents.			
Erosion on Spillway	Rock is missing and soil is exposed at top of spillway or outside slope.			
Gate/Fence Damage	Damage to gate/fence, including missing locks and hinges			

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

I.9 Dry Extended Detention Basin Inspection and Maintenance Checklist

Date: ____________________________ Work Order # ____________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: __________________________ Inspector(s): ______________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0, 1 or 2)†	Date Maintenance Performed	Comments or Action(s) Taken to Resolve Issue
General				
Appearance	Untidy, un-mown (if applicable)			
Vegetation	Access problems or hazards; dead or dying trees			
	Poisonous or nuisance vegetation or noxious weeds			
Insects	Insects such as wasps and hornets interfere with maintenance activities.			
Rodent Holes	Any evidence of rodent holes if facility is acting as a dam or berm, or any evidence of water piping through dam or berm via rodent holes			
Trash and Debris	Trash and debris > 5 cf/1,000 sf (one standard size garbage can).			
Pollutants	Any evidence of oil, gasoline, contaminants or other pollutants			
Inlet/Outlet Pipe	Inlet/Outlet pipe clogged with sediment and/or debris. Basin not draining.			
Erosion	Erosion of the basin's side slopes and/or scouring of the basin bottom that exceeds 2-inches, or where continued erosion is prevalent.			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0, 1 or 2)†	Date Maintenance Performed	Comments or Action(s) Taken to Resolve Issue
Piping	Evidence of or visible water flow through basin berm.			
Settlement of Basin Dike/Berm	Any part of these components that has settled 4-inches or lower than the design elevation, or inspector determines dike/berm is unsound.			
Overflow Spillway	Rock is missing and/or soil is exposed at top of spillway or outside slope.			
Sediment Accumulation in Basin Bottom	Sediment accumulations in basin bottom that exceeds the depth of sediment zone plus 6-inches.			
Tree or shrub growth	Trees > 4 ft in height with potential blockage of inlet, outlet or spillway; or potential future bank stability problems			
Debris Barriers (e.g., Trash Racks)				
Trash and Debris	Trash or debris that is plugging more than 20% of the openings in the barrier.			
Damaged/ Missing Bars	Bars are bent out of shape more than 3 inches.			
	Bars are missing or entire barrier missing.			
	Bars are loose and rust is causing 50% deterioration to any part of barrier.			
Inlet/Outlet Pipe	Debris barrier missing or not attached to pipe.			
Fencing				
Missing or broken parts	Any defect in the fence that permits easy entry to a facility.			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0, 1 or 2)†	Date Maintenance Performed	Comments or Action(s) Taken to Resolve Issue
Erosion	Erosion more than 4 inches high and 12-18 inches wide, creating an opening under the fence.			
Damaged Parts	Damage to gate/fence, posts out of plumb, or rails bent more than 6 inches.			
Deteriorating Paint or Protective Coating	Part or parts that have a rusting or scaling condition that has affected structural adequacy.			
Gates				
Damaged or missing member	Missing gate or locking devices, broken or missing hinges, out of plum more than 6 inches and more than 1 foot out of design alignment, or missing stretcher bar, stretcher bands, and ties.			

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.

I.10 Proprietary Device Inspection and Maintenance Checklist

Date: ________________________ Work Order # __________________

Type of Inspection: □ post-storm □ annual □ routine □ post-wet season □ pre-wet season

Facility: ______________________ Inspector(s): ___________________

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Refer to the manufacturer's instructions for maintenance/inspection requirements, below are generic guidelines to supplement manufacturer's recommendations.				
Underground Vault				
Sediment Accumulation on Media	Sediment depth exceeds 0.25-inches.			
Sediment Accumulation in Vault	Sediment depth exceeds 6-inches in first chamber.			
Trash/Debris Accumulation	Trash and debris accumulated on compost filter bed.			
Sediment in Drain Pipes or Cleanouts	When drain pipes, clean-outs, become full with sediment and/or debris.			
Damaged Pipes	Any part of the pipes that are crushed or damaged due to corrosion and/or settlement.			
Access Cover Damaged/Not Working	Cover cannot be opened; one person cannot open the cover using normal lifting pressure, corrosion/deformation of cover.			
Vault Structure Includes Cracks in Wall, Bottom, Damage to	Cracks wider than 1/2-inch or evidence of soil particles entering the structure through the cracks, or maintenance/inspection personnel determine that the vault is not structurally sound.			

Defect	Conditions When Maintenance Is Needed	Inspection Result (0,1, or 2) †	Date Maintenance Performed	Comments or Action(s) taken to resolve issue
Frame and/or Top Slab	Cracks wider than 1/2-inch at the joint of any inlet/outlet pipe or evidence of soil particles entering through the cracks.			
Baffles	Baffles corroding, cracking warping, and/or showing signs of failure as determined by maintenance/inspection person.			
Access Ladder Damaged	Ladder is corroded or deteriorated, not functioning properly, not securely attached to structure wall, missing rungs, cracks, or misaligned.			
Below Ground Cartridge Type				
Filter Media	Drawdown of water through the media takes longer than 1 hour and/or overflow occurs frequently.			
Short Circuiting	Flows do not properly enter filter cartridges.			

†Maintenance: Enter 0 if satisfactory, 1 if maintenance is needed and include WO#. Enter 2 if maintenance was performed same day.