Design Considerations

- Aesthetics
- Hydraulic Head

Targeted Constituents

✓	Sediment	■
✓	Nutrients	●
✓	Trash	■
✓	Metals	■
✓	Bacteria	▲
✓	Oil and Grease	■
✓	Organics	■

Legend ***(Removal Effectiveness)***

● Low ■ High

▲ Medium

Description

Stormwater media filters are usually two-chambered including a pretreatment settling basin and a filter bed filled with sand or other absorptive filtering media. As stormwater flows into the first chamber, large particles settle out, and then finer particles and other pollutants are removed as stormwater flows through the filtering media in the second chamber. There are a number of design variations including the Austin sand filter, Delaware sand filter, and multi-chambered treatment train (MCTT).

California Experience

Caltrans constructed and monitored five Austin sand filters, two MCTTs, and one Delaware design in southern California. Pollutant removal was very similar for each of the designs; however operational and maintenance aspects were quite different. The Delaware filter and MCTT maintain permanent pools and consequently mosquito management was a critical issue, while the Austin style which is designed to empty completely between storms was less affected. Removal of the top few inches of sand was required at 3 of the Austin filters and the Delaware filter during the third year of operation; consequently, sizing of the filter bed is a critical design factor for establishing maintenance frequency.

Advantages

- Relatively high pollutant removal, especially for sediment and associated pollutants.
- Widespread application with sufficient capture volume can provide significant control of channel erosion and enlargement caused by changes to flow frequency relationships resulting from the increase of impervious cover in a watershed.

Limitations

CASQA California Stormwater Quality Association

- More expensive to construct than many other BMPs.
- May require more maintenance that some other BMPs depending upon the sizing of the filter bed.
- Generally require more hydraulic head to operate properly (minimum 4 feet).
- High solids loads will cause the filter to clog.
- Work best for relatively small, impervious watersheds.
- Filters in residential areas can present aesthetic and safety problems if constructed with vertical concrete walls.
- Certain designs (e.g., MCTT and Delaware filter) maintain permanent sources of standing water where mosquito and midge breeding is likely to occur.

Design and Sizing Guidelines

- Capture volume determined by local requirements or sized to treat 85% of the annual runoff volume.
- Filter bed sized to discharge the capture volume over a period of 48 hours.
- Filter bed 18 inches thick above underdrain system.
- Include energy dissipation in the inlet design to reduce resuspension of accumulated sediment.
- A maintenance ramp should be included in the design to facilitate access to the sedimentation and filter basins for maintenance activities (particularly for the Austin design).
- Designs that utilize covered sedimentation and filtration basins should be accessible to vector control personnel via access doors to facilitate vector surveillance and controlling the basins if needed.

Construction/Inspection Considerations

- Tributary area should be completely stabilized before media is installed to prevent premature clogging.

Performance

The pollutant removal performance of media filters and other stormwater BMPs is generally characterized by the percent reduction in the influent load. This method implies a relationship between influent and effluent concentrations. For instance, it would be expected that a device that is reported to achieve a 75% reduction would have an effluent concentration equal to 25% of the influent concentrations. Recent work in California (Caltrans, 2002) on various sand filter designs indicates that this model for characterizing performance is inadequate. Figure 4 presents a graph relating influent and effluent TSS concentrations for the Austin full sedimentation design.

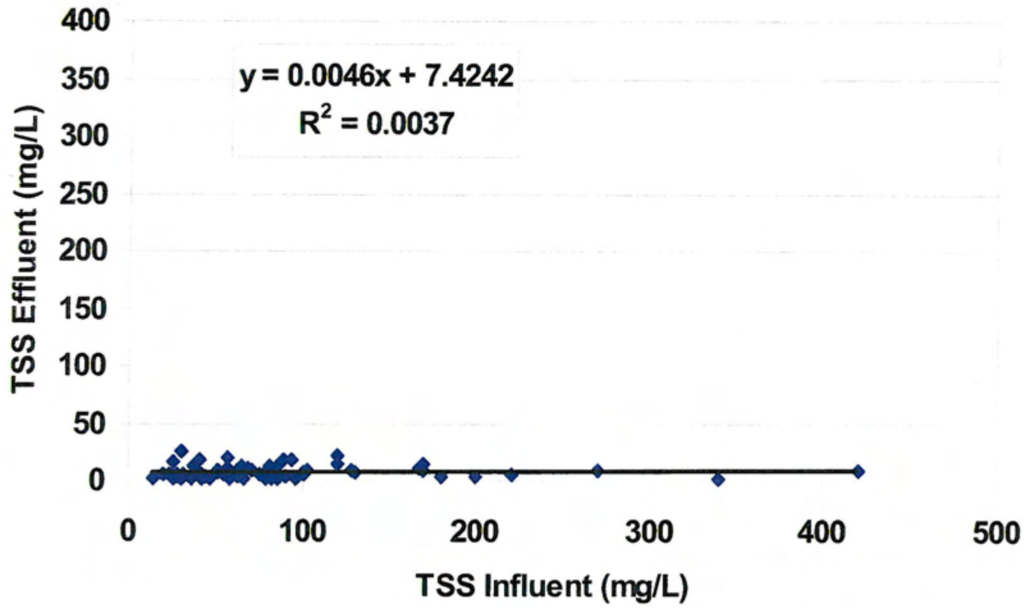


Figure 4
Comparison of Influent and Effluent Concentrations for TSS

It is clearly evident that the effluent concentration is relative constant and independent of influent concentration. Consequently, the performance is more accurately characterized by the effluent concentration, which is about 7.5 mg/L. Constant effluent concentrations also are observed for all other particle related constituents such as particulate metals (total - dissolved) and particulate phosphorus.

The small uncertainty in the estimate of the mean effluent concentration highlights the very consistent effluent quality for TSS produced by sand filters. In addition, it demonstrates that a calculated percent reduction for TSS and other constituents with similar behavior for Austin sand filters is a secondary characteristic of the device and depends primarily on the specific influent concentrations observed. The distinction between a constant effluent quality and a percent reduction is extremely important to recognize if the results are to be used to estimate effluent quality from sand filters installed at other sites with different influent concentrations or for estimating compliance with water quality standards for storms with high concentrations of particulate constituents.

If the conventionally derived removal efficiency (90%) were used to estimate the TSS concentrations in the treated runoff from storms with high influent concentrations, the estimated effluent concentration would be too high. For instance, the storm with the highest observed influent concentration (420 mg/L) would be expected to have a concentration in the treated runoff of 42 mg/L, rather than the 10 mg/L that was measured. In fact, the TSS effluent concentrations for all events with influent concentrations greater than 200 mg/L were 10 mg/L or less.

The stable effluent concentration of a sand filter under very different influent TSS concentrations implies something about the properties of the influent particle size distribution. If one assumes that only the smallest size fraction can pass through the filter, then the similarity in effluent concentrations suggests that there is little difference in the total mass of the smallest sized particles even when the total TSS concentration varies greatly. Further, the difference in TSS concentration must then be caused by changes in the relative amount of the larger size fractions. Further research is necessary to determine the range of particle size that is effectively removed in the filter and the portion of the size fraction of suspended solids that it represents in urban stormwater.

Sand filters are effective stormwater management practices for pollutant removal. Conventional removal rates for all sand filters and organic filters are presented in Table 1. With the exception of nitrates, which are always exported from filtering systems because of the conversion of ammonia and organic nitrogen to nitrate, they perform relatively well at removing pollutants.

Table 1 Sand filter removal efficiencies (percent)

	Sand Filter (Glick et al, 1998)	Compost Filter System		Multi-Chamber Treatment Train		
		Stewart, 1992	Leif, 1999	Pitt et al., 1997	Pitt, 1996	Greb et al., 1998
TSS	89	95	85	85	83	98
TP	59	41	4	80	-	84
TN	17	-	-	-	-	-
Nitrate	-76	-34	-95	-	14	-
Metals	72-86	61-88	44-75	65-90	91-100	83-89
Bacteria	65	-	-	-	-	-

From the few studies available, it is difficult to determine if organic filters necessarily have higher removal efficiencies than sand filters. The MCTT may have high pollutant removal for some constituents, although an evaluation of these devices by the California Department of Transportation indicated no significant difference for most conventional pollutants.

In addition to the relatively high pollutant removal in media filters, these devices, when sized to capture the channel forming storm volume, are highly effective at attenuating peak flow rates and reducing channel erosion.

Siting Criteria

In general, sand filters are preferred over infiltration practices, such as infiltration trenches, when contamination of groundwater with conventional pollutants is of concern. This usually occurs in areas where underlying soils alone cannot treat runoff adequately - or ground water tables are high. In most cases, sand filters can be constructed with impermeable basin or chamber bottoms, which help to collect, treat, and release runoff to a storm drainage system or directly to surface water with no contact between contaminated runoff and groundwater. In regions where evaporation exceeds rainfall and a wet pond would be unlikely to maintain the required permanent pool, a sand filtration system can be used.

The selection of a sand filter design depends largely on the drainage area's characteristics. For example, the Washington, D.C. and Delaware sand filter systems are well suited for highly impervious areas where land available for structural controls is limited, since both are installed underground. They have been used to treat runoff from parking lots, driveways, loading docks, service stations, garages, airport runways/taxiways, and storage yards. The Austin sand filtration system is more suited for large drainage areas that have both impervious and pervious surfaces. This system is located at grade and is used to treat runoff from any urban land use.

It is challenging to use most sand filters in very flat terrain because they require a significant amount of hydraulic head (about 4 feet), to allow flow through the system. One exception is the perimeter sand filter, which can be applied with as little as 2 feet of head.

Sand filters are best applied on relatively small sites (up to 25 acres for surface sand filters and closer to 2 acres for perimeter or underground filters). Filters have been used on larger drainage areas, of up to 100 acres, but these systems can clog when they treat larger drainage areas unless adequate measures are provided to prevent clogging, such as a larger sedimentation chamber or more intensive regular maintenance.

When sand filters are designed as a stand-alone practice, they can be used on almost any soil because they can be designed so that stormwater never infiltrates into the soil or interacts with the ground water. Alternatively, sand filters can be designed as pretreatment for an infiltration practice, where soils do play a role.

Additional Design Guidelines

Pretreatment is a critical component of any stormwater management practice. In sand filters, pretreatment is achieved in the sedimentation chamber that precedes the filter bed. In this chamber, the coarsest particles settle out and thus do not reach the filter bed. Pretreatment reduces the maintenance burden of sand filters by reducing the potential for these sediments to clog the filter. When pretreatment is not provided designers should increase the size of the filter area to reduce the clogging potential. In sand filters, designers should select a medium sand as the filtering medium. A fine aggregate (ASTM C-33) that is intended for use in concrete is commonly specified.

Many guidelines recommend sizing the filter bed using Darcy's Law, which relates the velocity of fluids to the hydraulic head and the coefficient of permeability of a medium. The resulting equation, as derived by the city of Austin, Texas, (1996), is

$$Af = WQV\, d/[k\, t\, (h+d)]$$

Where:

Af = area of the filter bed (ft^2);

d = depth of the filter bed (ft; usually about 1.5 feet, depending on the design);

k = coefficient of permeability of the filtering medium (ft/day);

t = time for the water quality volume to filter through the system (days; usually assumed to be 1.67 days); and

h = average water height above the sand bed (ft; assumed to be one-half of the maximum head).

Typical values for k, as assembled by CWP (1996), are shown in Table 2.

Table 2 Coefficient of permeability values for stormwater filtering practices (CWP, 1996)

Filter Medium	Coefficient of Permeability (ft/day)
Sand	3.5
Peat/Sand	2.75
Compost	8.7

The permeability of sand shown in Table 2 is extremely conservative, but is widely used since it is incorporated in the design guidelines of the City of Austin. When the sand is initially installed, the permeability is so high (over 100 ft/d) that generally only a portion of the filter area is required to infiltrate the entire volume, especially in a "full sedimentation" Austin design where the capture volume is released to the filter basin over 24 hours.

The preceding methodology results in a filter bed area that is oversized when new and the entire water quality volume is filtered in less than a day with no significant height of water on top of the sand bed. Consequently, the following simple rule of thumb is adequate for sizing the filter area. If the filter is preceded by a sedimentation basin that releases the water quality volume (WQV) to the filter over 24 hours, then

$$Af = WQV/18$$

If no pretreatment is provided then the filter area is calculated more conservatively as:

$$Af = WQV/10$$

Typically, filtering practices are designed as "off-line" systems, meaning that during larger storms all runoff greater than the water quality volume is bypassed untreated using a flow splitter, which is a structure that directs larger flows to the storm drain system or to a stabilized channel. One exception is the perimeter filter; in this design, all flows enter the system, but larger flows overflow to an outlet chamber and are not treated by the practice.

The Austin design variations are preferred where there is sufficient space, because they lack a permanent pool, which eliminates vector concerns. Design details of this variation are summarized below.

Summary of Design Recommendations

(1) Capture Volume - The facility should be sized to capture the required water quality volume, preferably in a separate pretreatment sedimentation basin.

(2) Basin Geometry – The water depth in the sedimentation basin when full should be at least 2 feet and no greater than 10 feet. A fixed vertical sediment depth marker should be installed in the sedimentation basin to indicate when 20% of the basin volume has been lost because of sediment accumulation. When a pretreatment sedimentation basin is provided the minimum average surface area for the sand filter (Af) is calculated from the following equation:

$$Af = WQV/18$$

If no pretreatment is provided then the filter area is calculated as:

$$Af = WQV/10$$

(3) Sand and Gravel Configuration - The sand filter is constructed with 18 inches of sand overlying 6 inches of gravel. The sand and gravel media are separated by permeable geotextile fabric and the gravel layer is situated on geotextile fabric. Four-inch perforated PVC pipe is used to drain captured flows from the gravel layer. A minimum of 2 inches of gravel must cover the top surface of the PVC pipe. Figure 5 presents a schematic representation of a standard sand bed profile.

(4) Sand Properties – The sand grain size distribution should be comparable to that of "washed concrete sand," as specified for fine aggregate in ASTM C-33.

(5) Underdrain Pipe Configuration – In an Austin filter, the underdrain piping should consist of a main collector pipe and two or more lateral branch pipes, each with a minimum diameter of 4 inches. The pipes should have a minimum slope of 1% (1/8 inch per foot) and the laterals should be spaced at intervals of no more than 10 feet. There should be no fewer than two lateral branch pipes. Each individual underdrain pipe should have a cleanout access location. All piping is to be Schedule 40 PVC. The maximum spacing between rows of perforations should not exceed 6 inches.

(6) Flow Splitter - The inflow structure to the sedimentation chamber should incorporate a flow-splitting device capable of isolating the capture volume and bypassing the 25-year peak flow around the facility with the sedimentation/filtration pond full.

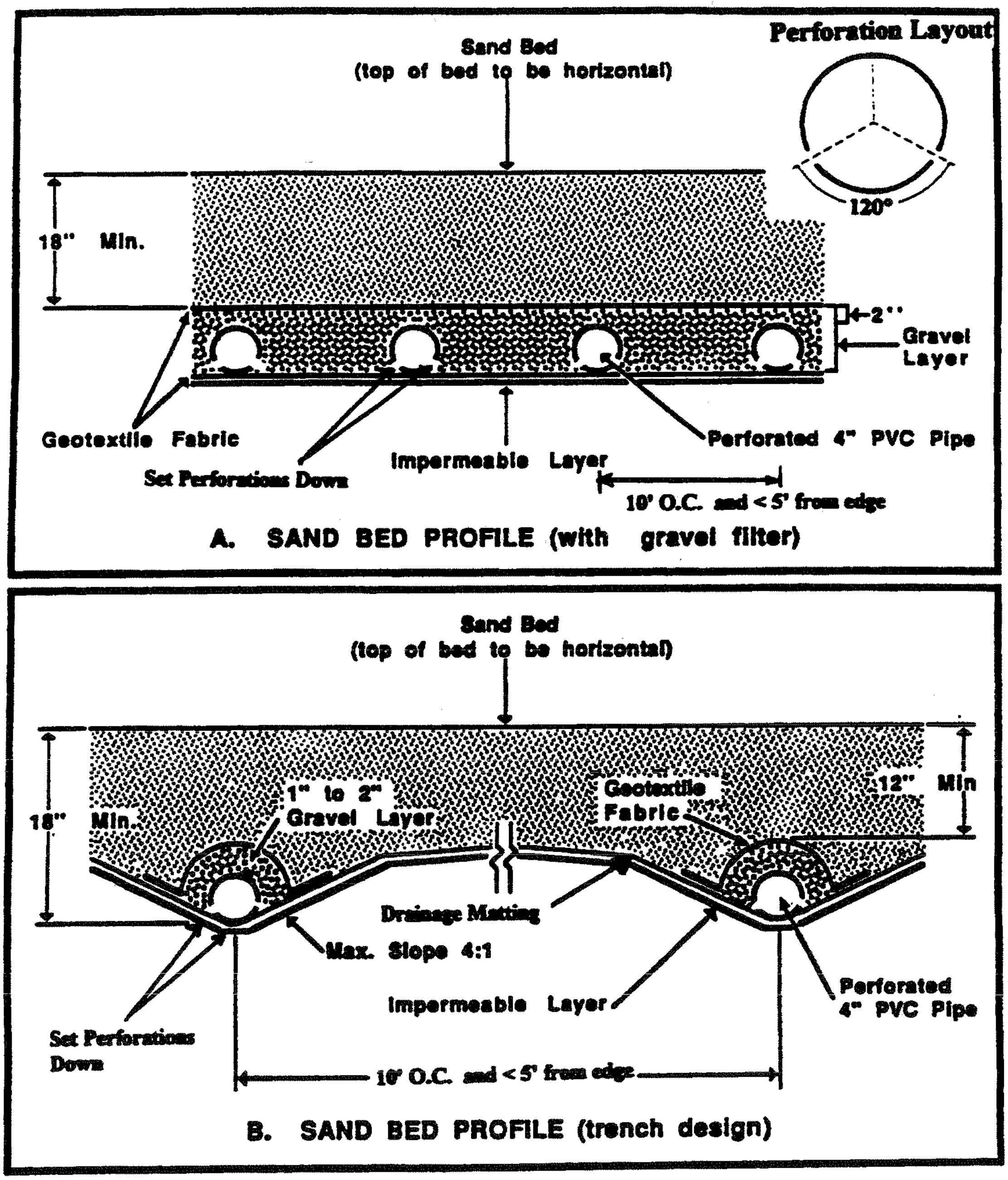


Figure 5
Schematic of Sand Bed Profile

(7) Basin Inlet – Energy dissipation is required at the sedimentation basin inlet so that flows entering the basin should be distributed uniformly and at low velocity in order to prevent resuspension and encourage quiescent conditions necessary for deposition of solids.

(8) Sedimentation Pond Outlet Structure - The outflow structure from the sedimentation chamber should be (1) an earthen berm; (2) a concrete wall; or (3) a rock gabion. Gabion outflow structures should extend across the full width of the facility such that no short-circuiting of flows can occur. The gabion rock should be 4 inches in diameter. The

receiving end of the sand filter should be protected (splash pad, riprap, etc.) such that erosion of the sand media does not occur. When a riser pipe is used to connect the sedimentation and filtration basins (example in Figure 6), a valve should be included to isolate the sedimentation basin in case of a hazardous material spill in the watershed. The control for the valve must be accessible at all times, including when the basin is full. The riser pipe should have a minimum diameter of 6 inches with four 1-inch perforations per row. The vertical spacing between rows should be 4 inches (on centers).

(9) Sand Filter Discharge – If a gabion structure is used to separate the sedimentation and filtration basins, a valve must installed so that discharge from the BMP can be stopped in case runoff from a spill of hazardous material enters the sand filter. The control for the valve must be accessible at all times, including when the basin is full.

Maintenance

Even though sand filters are generally thought of as one of the higher maintenance BMPs, in a recent California study an average of only about 49 hours a year were required for field activities. This was less maintenance than was required by extended detention basins serving comparable sized catchments. Most maintenance consists of routine removal of trash and debris, especially in Austin sand filters where the outlet riser from the sedimentation basin can become clogged.

Most data (i.e. Clark, 2001) indicate that hydraulic failure from clogging of the sand media occurs before pollutant breakthrough. Typically, only the very top of the sand becomes clogged while the rest remains in relative pristine condition as shown in Figure 7. The rate of clogging has been related to the TSS loading on the filter bed (Urbonas, 1999); however, the data are quite variable. Empirical observation of sites treating urban and highway runoff indicates that clogging of the filter occurs after 2 – 10 years of service. Presumably, this is related to differences in the type and amount of sediment in the catchment areas of the various installations. Once clogging occurs the top 2 – 3 inches of filter media is removed, which restores much, but not all, of the lost permeability. This removal of the surface layer can occur several times before the entire filter bed must be replaced. The cost of the removal of the surface layer is not prohibitive, generally ranging between $2,000 (EPA Fact Sheet) and $4,000 (Caltrans, 2002) depending on the size of the filter.

Media filters can become a nuisance due to mosquito and midge breeding in certain designs or if not regularly maintained. "Wet" designs (e.g., MCTT and Delaware filter) are more conducive to vectors than others (e.g., Austin filters) because they maintain permanent sources of standing water where breeding is likely to occur. Caltrans successfully excluded mosquitoes and midges from accessing the permanent water in the sedimentation basin of MCTT installations through use of a tight-fitting aluminum cover to seal vectors out. However, typical wet designs may require routine inspections and treatments by local mosquito and vector control agencies to suppress mosquito production. Vector habitats may also be created in "dry" designs when media filters clog, and/or when features such as level spreaders that hold water over 72 hours are included in the installation. Dry designs such as Austin filters should dewater completely (recommended 72 hour residence time or less) to prevent creating mosquito and other vector habitats. Maintenance efforts to prevent vector breeding in dry designs will need to focus on basic housekeeping practices such as removal of debris accumulations and vegetation management (in filter media) to prevent clogs and/or pools of standing water.

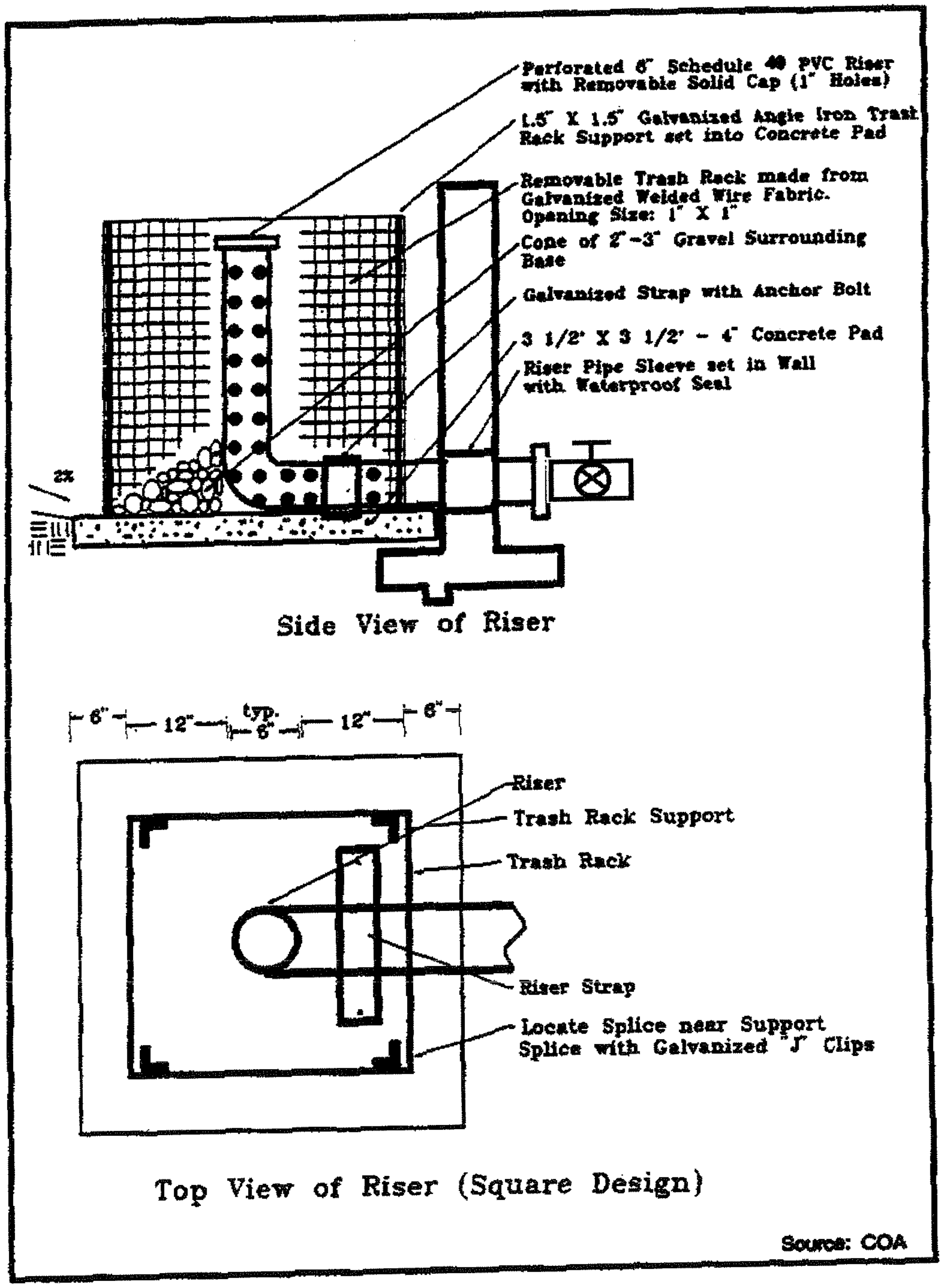


Figure 6
Detail of Sedimentation Riser Pipe

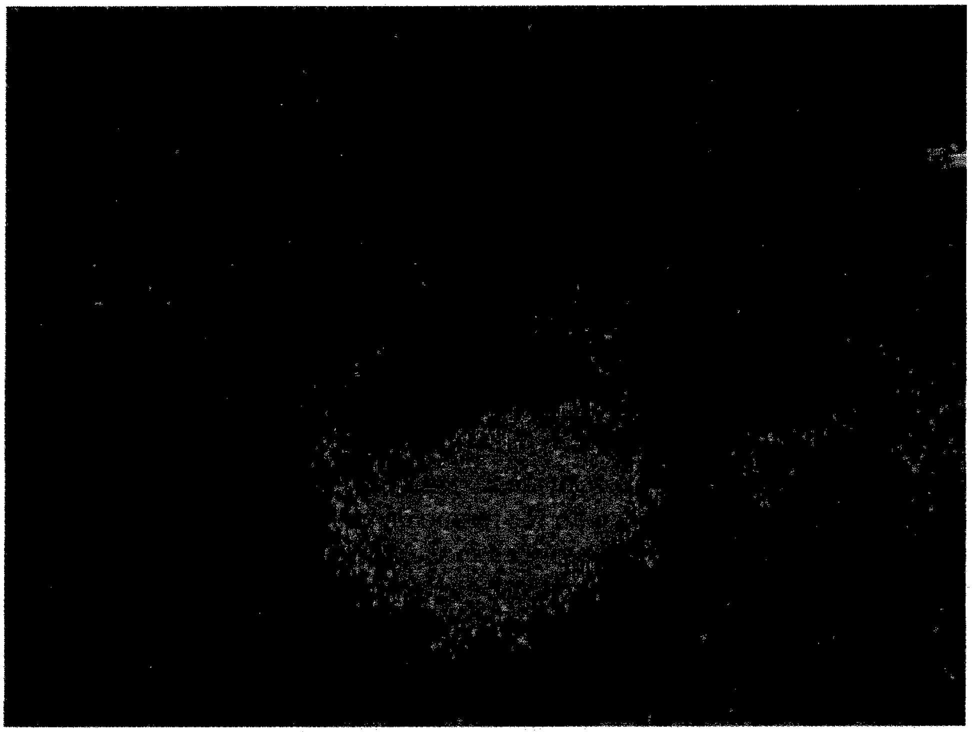


Figure 7
Formation of Clogging Crust on Filter Bed

Recommended maintenance activities and frequencies include:

- Inspections semi-annually for standing water, sediment, trash and debris, and to identify potential problems.
- Remove accumulated trash and debris in the sedimentation basin, from the riser pipe, and the filter bed during routine inspections.
- Inspect the facility once during the wet season after a large rain event to determine whether the facility is draining completely within 72 hr.
- Remove top 50 mm (2 in.) of sand and dispose of sediment if facility drain time exceeds 72 hr. Restore media depth to 450 mm (18 in.) when overall media depth drops to 300 mm (12 in.).
- Remove accumulated sediment in the sedimentation basin every 10 yr or when the sediment occupies 10 percent of the basin volume, whichever is less.

Cost

Construction Cost

There are few consistent published data on the cost of sand filters, largely because, with the exception of Austin, Texas, Alexandria, Virginia, and Washington, D.C., they have not been widely used. Furthermore, filters have such varied designs that it is difficult to assign a cost to filters in general. A study by Brown and Schueler (1997) was unable to find a statistically valid relationship between the volume of water treated in a filter and the cost of the practice. The EPA filter fact sheet indicates a cost for an Austin sand filter at $18,500 (1997 dollars) for a 0.4 hectare- (1 acre-)

drainage area. However, the same design implemented at a 1.1 ha site by the California Department of Transportation, cost $240,000. Consequently, there is a tremendous uncertainty about what the average construction cost might be.

It is important to note that, although underground and perimeter sand filters can be more expensive than surface sand filters, they consume no surface space, making them a relatively cost-effective practice in ultra-urban areas where land is at a premium.

Given the number of facilities installed in the areas that promote their use it should be possible to develop fairly accurate construction cost numbers through a more comprehensive survey of municipalities and developers that have implemented these filters.

Maintenance Cost

Annual costs for maintaining sand filter systems average about 5 percent of the initial construction cost (Schueler, 1992). Media is replaced as needed, with the frequency correlated with the solids loading on the filter bed. Currently the sand is being replaced in the D.C. filter systems about every 2 years, while an Austin design might last 3-10 years depending on the watershed characteristics. The cost to replace the gravel layer, filter fabric and top portion of the sand for D.C. sand filters is approximately $1,700 (1997 dollars).

Caltrans estimated future maintenance costs for the Austin design, assuming a device sized to treat runoff from approximately 4 acres. These estimates are presented in Table 3 and assume a fully burdened hourly rate of $44 for labor. This estimate is somewhat uncertain, since complete replacement of the filter bed was not required during the period that maintenance costs were recorded.

Table 3 Expected Annual Maintenance Costs for an Austin Sand Filter

Activity	Labor Hours	Equipment and Materials ($)	Cost
Inspections	4	0	176
Maintenance	36	125	1,706
Vector Control	0	0	0
Administration	3	0	132
Direct Costs	-	888	888
Total	43	$1,013	**$2,902**

References and Sources of Additional Information

Barton Springs/Edwards Aquifer Conservation District. 1996. *Final Report: Enhanced Roadway Runoff Best Management Practices.* City of Austin, Drainage Utility, LCRA, TDOT. Austin, TX. 200 pp.

Bell, W., L. Stokes, L.J. Gavan, and T.N. Nguyen. 1995. *Assessment of the Pollutant Removal Efficiencies of Delaware Sand Filter BMPs.* Final Report. Department of Transportation and

Environmental Services. Alexandria, VA. 140 pp. Also in Performance of Delaware Sand Filter Assessed. Watershed Protection Techniques. Center for Watershed Protection. Fall 1995. Vol. 2(1): 291–293.

Brown, W., and T. Schueler. 1997. *The Economics of Stormwater BMPs in the Mid-Atlantic Region.* Prepared for the Chesapeake Research Consortium, Edgewater, MD, by the Center for Watershed Protection, Ellicott City, MD.

Caltrans, 2002, *Proposed Final Report: BMP Retrofit Pilot Program*, California Dept. of Transportation Report CTSW-RT-01-050, Sacramento, CA.

Center for Watershed Protection (CWP). 1996. *Design of Stormwater Filtering Systems.* Prepared for the Chesapeake Research Consortium, Solomons, MD, and U.S. EPA Region 5, Chicago, IL, by the Center for Watershed Protection, Ellicott City, MD.

Center for Watershed Protection (CWP). 1997. Multi-Chamber Treatment Train developed for stormwater hot spots. *Watershed Protection Techniques* 2(3):445–449.

City of Austin, TX. 1990. *Removal Efficiencies of Stormwater Control Structures.* Final Report. Environmental Resource Management Division. 36 p. Also in: Developments in Sand Filter Technology to Improve Stormwater Runoff Quality. Watershed Protection Techniques. Center for Watershed Protection. Summer 1994. Vol. 1(2): 47–54.

City of Austin, TX. 1996. *Design of Water Quality Controls.* City of Austin, TX.

Clark, S.E., 2000, Urban Stormwater Filtration: Optimization of Design Parameters and a Pilot-Scale Evaluation, Ph.D. Dissertation, University of Alabama at Birmingham.

CSF Treatment Systems, Inc. (CSF). 1996. *Stormwater management promotional brochure.* CSF Treatment Systems, Inc., Portland, OR.

Curran, T. 1996. Peat Sand Efficiency Calculations for McGregor Park. Unpublished data. Lower Colorado River Authority. Austin, TX.

Galli, F. 1990. Peat-Sand Filters: *A Proposed Stormwater Management Practice for Urban Areas.* Metropolitan Washington Council of Governments, Washington, DC.

Glick, Roger, Chang, George C., and Barrett, Michael E., 1998, Monitoring and evaluation of stormwater quality control basins, in *Watershed Management: Moving from Theory to Implementation*, Denver, CO, May 3-6, 1998, pp. 369 – 376.

Greb, S., S. Corsi, and R. Waschbush. 1998. Evaluation of Stormceptor© and Multi-Chamber Treatment Train as Urban Retrofit Strategies. Presented at Retrofit Opportunities for Water Resource Protection in Urban Environments, A National Conference. The Westin Hotel, Chicago, IL, February 10–12, 1998.

Harper, H., and J. Herr. 1993. *Treatment Efficiency of Detention With Filtration Systems.* Environmental Research and Design, Inc. Final Report Submitted to Florida Department of Environmental Regulation. Orlando, FL. 164 pp.

Horner, R.R. and Horner, C.R., 1999, Performance of a Perimeter ("Delaware") Sand Filter in Treating Stormwater Runoff from a Barge Loading Terminal. *Proc. of the Comprehensive Stormwater and Aquatic Ecosystem Management Conf.*, Auckland, N.Z., Feb. 1999, pp. 183-192.

Horner, R.R., and C.R. Horner. 1995. *Design, Construction and Evaluation of a Sand Filter Stormwater Treatment System.* Part II. Performance Monitoring. Report to Alaska Marine Lines, Seattle, WA. 38 p. Also in Performance of Delaware Sand Filter Assessed. Watershed Protection Techniques. Center for Watershed Protection. Fall 1995. Vol. 2(1): 291–293.

Keblin, Michael V., Barrett, Michael E., Malina, Joseph F., Jr., Charbeneau, Randall J, 1998, *The Effectiveness of Permanent Highway Runoff Controls: Sedimentation/Filtration Systems,* Research Report 2954-1, Center for Transportation Research, University of Texas at Austin.

King County, Washington, Department of Natural Resources. 2000. *King County Surface Water Design Manual.* http://splash.metrokc.gov/wlr/dss/manual.htm.Last updated March 6, 2000. Accessed January 5, 2001.

Leif, T. 1999. *Compost Stormwater Filter Evaluation.* Snohomish County, Washington, Department of Public Works, Everett, WA.

Maryland Department of the Environment (MDE). 2000. *Maryland Stormwater Design Manual.* http://www.mde.state.md.us/environment/wma/stormwatermanual. Accessed May 22, 2001.

Metzger, M. E., D. F. Messer, C. L. Beitia, C. M. Myers, and V. L. Kramer. 2002. The fvBMPs. Stormwater 3(2): 24-39.

Pitt, R. 1996. The Control of Toxicants at Critical Source Areas. Presented at the ASCE/Engineering Foundation Conference, Snowbird, UT, August 1996.

Pitt, R., M. Lilburn, and S. Burian. 1997. *Storm Drainage Design for the Future: Summary of Current U.S. EPA Research.* American Society of Civil Engineers Technical Conference, Gulf Shores, AL, July 1997.

Robertson, B., R. Pitt, A. Ayyoubi, and R. Field. 1995. A Multi-Chambered Stormwater Treatment Train. In *Proceedings of the Engineering Foundation Conference: Stormwater NPDES-Related Monitoring Needs, Mt. Crested Butte, Colorado, August 7–12, 1994,* American Society of Civil Engineers, New York, New York.

Schueler, T. 1994. Developments in sand filter technology to improve stormwater runoff quality. *Watershed Protection Techniques* 1(2):47–54.

Schueler, T. 1997. Comparative Pollutant Removal Capability of Urban BMPs: A Reanalysis. *Watershed Protection Techniques* 2(4):515–520.

Stewart, W. 1992. *Compost Stormwater Treatment System.* W&H Pacific Consultants. Draft Report. Portland, OR. Also in Innovative Leaf Compost System Used to Filter Runoff at Small Sites in the Northwest. *Watershed Protection Techniques.* Center for Watershed Protection. February 1994. Vol. 1(1): 13–14.

Urbonas, B.R, 1999, Design of a sand filter for stormwater quality enhancement, Water Environment Research, V. 71, No. 1, pp. 102-113.

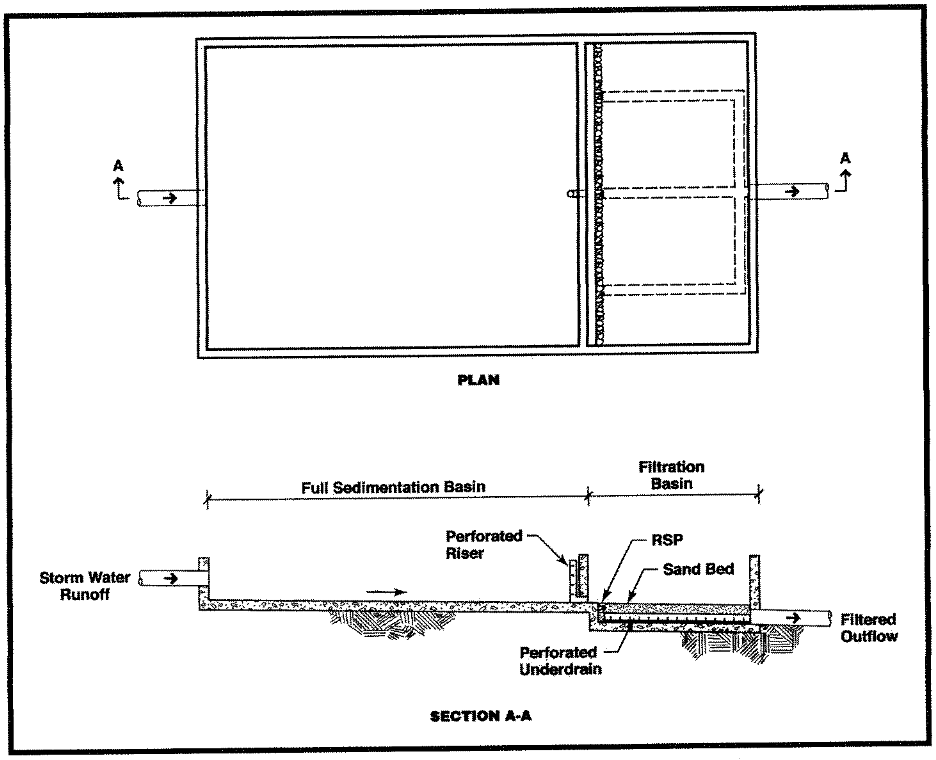
U.S. EPA, 1999, Stormwater Technology Fact Sheet: Sand Filters, Report EPA 832-F-99-007 http://www.epa.gov/owm/mtb/sandfltr.pdf, Office of Water, Washington, DC

Washington State Department of Ecology (DOE). 1992. *Stormwater Management Manual for the Puget Sound Basin*, Washington State Department of Ecology, Olympia, WA.

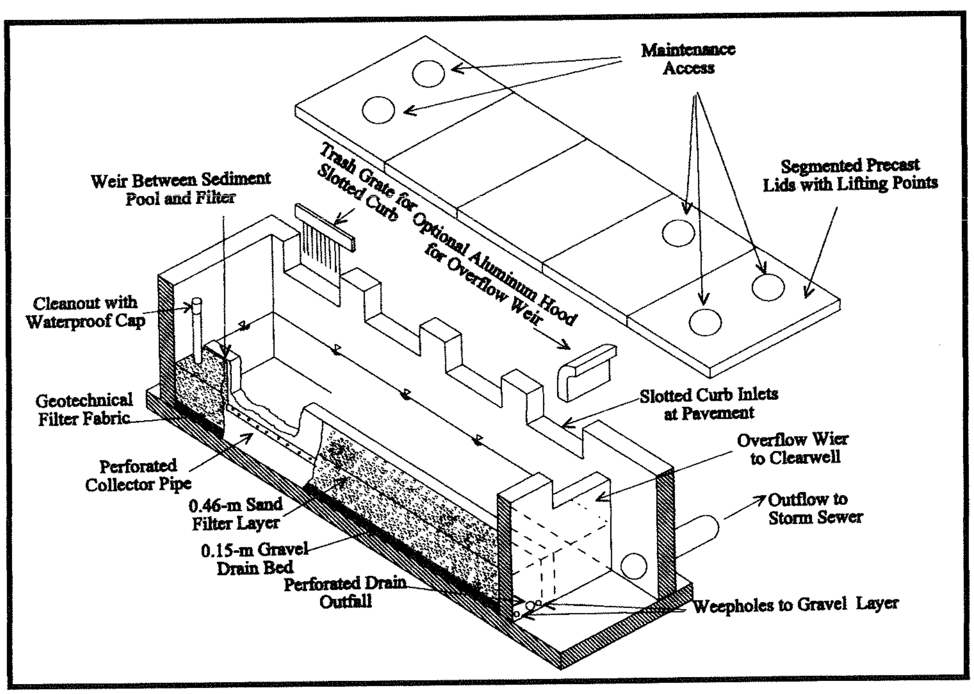
Watershed Management Institute (WMI). 1997. *Operation, Maintenance, and Management of Stormwater Management Systems*. Prepared for U.S. EPA Office of Water, Washington, DC, by Watershed Management Institute.

Welborn, C., and J. Veenhuis. 1987. *Effects of Runoff Controls on the Quantity and Quality of Urban Runoff in Two Locations in Austin, TX*. USGS Water Resources Investigations Report. 87–4004. 88 pp.

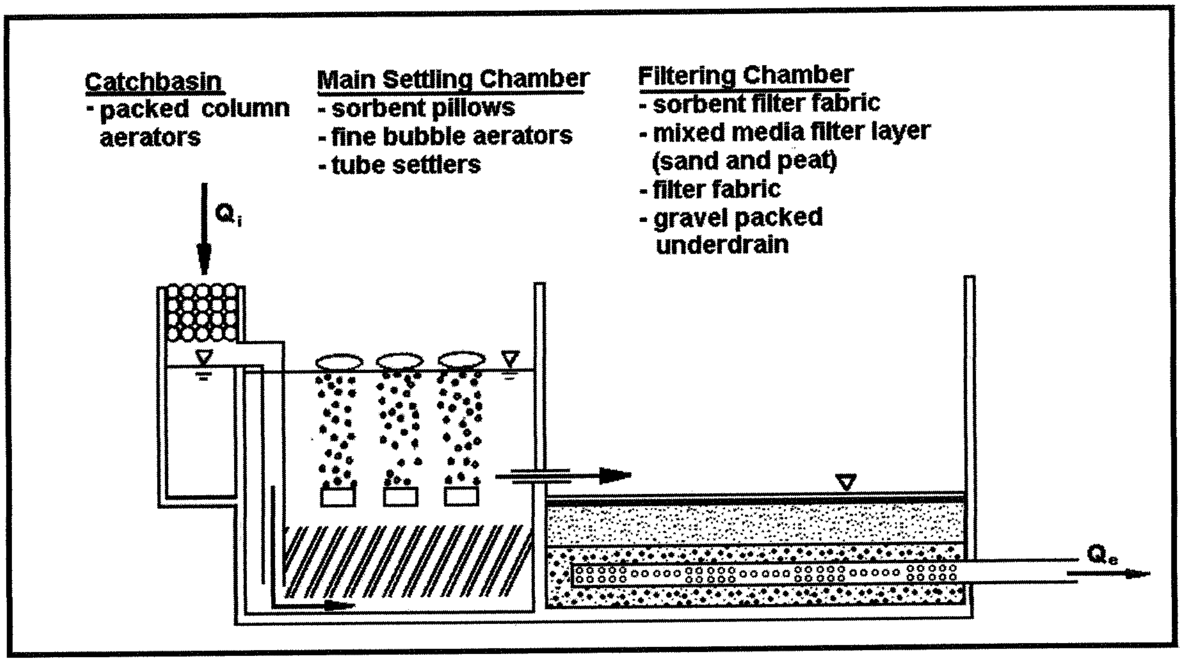
Young, G.K., et al., 1996, *Evaluation and Management of Highway Runoff Water Quality*, Publication No. FHWA-PD-96-032, U.S. Department of Transportation, Federal Highway Administration, Office of Environment and Planning.



Schematic of the "Full Sedimentation" Austin Sand Filter



Schematic of a Delaware Sand Filter (Young et al., 1996)



Schematic of a MCTT (Robertson et al., 1995)

Appendix E

Hydromodification Analysis

Prepared for

SunRanch Capital Partners, LLC and USA Portola Properties, LLC
610 West Ash Street, Suite 1500
San Diego, CA 92101

Portola Center

Hydromodification Analysis

Prepared by

Geosyntec consultants

engineers | scientists | innovators

1111 Broadway, 6th Floor
Oakland, California 94607

WW1536 & WW1547

15 January 2013

Final Report

Geosyntec
consultants

TABLE OF CONTENTS

1. INTRODUCTION 1

2. BACKGROUND 1

3. ANALYSIS 2
   - 3.1 Step 1: Characterize Site Specific Hydrologic Conditions 3
     - 3.1.1 Soil Type 4
     - 3.1.2 Drainage Catchments 4
     - 3.1.3 Vegetation Cover 5
     - 3.1.4 Impervious Cover 6
     - 3.1.5 Overland Slope 6
   - 3.2 Step 2: Locate Structural BMPs and Select Type 6
   - 3.3 Step 3: Select Hydrologic Modeling Parameters 7
   - 3.4 Step 4: Define the Flow Range of Interest ($0.1Q_2$, Q_5, and Q_{10}) 8
   - 3.5 Step 5: Select Configuration of Structural BMPs 8
     - 3.5.1 Hydraulic Outlet Configuration 9
     - 3.5.2 Geometric Configuration 9
     - 3.5.3 Infiltration Rate 9
   - 3.6 Step 6: Iteratively Size the BMP Footprints to Meet the IHC 9
   - 3.7 Step 7: Iterate BMP Locations, Types, Configurations, and Sizes to Best Meet Proposed Layout 10

4. CONCLUSION 10

5. REFERENCES 11

LIST OF TABLES

Table 1: Summary of Pre- and Post-Development Hydrologic Conditions

Table 2: BMP Tributary Areas

Table 3: Pre-Development Sub-Catchment Parameters

Table 4: Post-Development Sub-Catchment Parameters

Table 5: SWMM Parameters

Table 6: Partial Duration Series Results for Pre-Development Simulation

Table 7: Proposed BMP Hydraulic Outlet Configuration

Table 8: Proposed BMP Storage Volume

LIST OF FIGURES

Figure 1a: Modeled SWMM Network – Outfall A

Figure 1b: Modeled SWMM Network – Outfall B

Figure 1c: Modeled SWMM Network – Outfall C

Figure 1d: Modeled SWMM Network – Outfall D

Figure 2: Stage-Storage-Area Relationship - BMP 9

Figure 3a: Flow Duration Curve Comparison – Outfall A

Figure 3b: Flow Duration Curve Comparison – Outfall B

Figure 3c: Flow Duration Curve Comparison – Outfall C

Figure 3d: Flow Duration Curve Comparison – Outfall D

Figure 4a: Flow Frequency Comparison – Outfall A

Figure 4b: Flow Frequency Comparison – Outfall B

Figure 4c: Flow Frequency Comparison – Outfall C

Figure 4d: Flow Frequency Comparison – Outfall D

LIST OF EXHIBITS

Exhibit 1: Vicinity Map

Exhibit 2: Soil Characterization

Exhibit 3: Pre-Development Drainage Delineation

Exhibit 4: Post-Development Drainage Delineation and BMP Locations

1. INTRODUCTION

This report describes the analyses performed for sizing hydromodification control facilities (commonly referred to as structural BMPs) for the Portola Center project in Lake Forest, California, located at the intersection of Glenn Ranch Road and Saddleback Ranch Road (Exhibit 1). The analysis performed includes proposed new development for Tentative Tracts 15353 and 17300. Hydromodification control facilities are required to comply with the Interim Hydromodification Criteria (IHC) defined in the South Orange County MS4 permit. The method of analysis utilized herein is the Site Specific System (System Based) approach, which is an accepted alternative to the South Orange County Hydromodification BMP Sizing Tool (Interim Sizing Tool) (OC Watersheds 2011). The System Based approach is tailored to site-specific conditions using local data, in contrast to the Interim Sizing Tool which necessarily relies on generalized data for South Orange County. The goal is to design hydromodification facilities that are appropriately sized for the Portola Center project.

2. BACKGROUND

The South Orange County MS4 Permit (Order No. R9-2009-0002) defines hydromodification as the "change in the natural watershed hydrologic processes and runoff characteristics (i.e., interception, infiltration, overland flow, interflow and groundwater flow) caused by urbanization or other land use changes that result in increased stream flows and sediment transport." Unless managed, hydromodification can cause channel erosion, sedimentation, planform migration, alteration to baseflow, or changes in bed material composition. Such geomorphic impacts also may impair beneficial uses of the stream or lead to biological impacts to streams. To prevent hydromodification impacts, priority development projects discharging to streams potentially sensitive to hydromodification, which includes the Portola Center project[1], must comply with the following Interim Hydromodification Criteria (IHC) defined in the MS4 Permit:

Within one year of this Order each Copermittee must ensure that all Priority Development Projects are implementing the following criteria by comparing the pre-development (naturally occurring) and post-project flow rates and durations using a

[1] Runoff from the Portola Center site discharges to tributaries of Aliso Creek. These are potentially sensitive channels per the IHC.

continuous simulation hydrologic model such as US EPA's Hydrograph Simulation Program-Fortran (HSPF):

(a) For flow rates from 10 percent of the 2-year storm event to the 5 year storm event, the post-project peak flows shall not exceed predevelopment (naturally occurring) peak flows.

(b) For flow rates from the 5 year storm event to the 10 year storm event, the post-project peak flows may exceed pre-development (naturally occurring) flows by up to 10 percent for a 1-year frequency interval.

The interim hydromodification criteria do not apply to Priority Development Projects where the project discharges (1) storm water runoff into underground storm drains discharging directly to bays or the ocean, or (2) storm water runoff into conveyance channels whose bed and bank are concrete lined all the way from the point of discharge to ocean waters, enclosed bays, estuaries, or water storage reservoirs and lakes.

The IHC is not limited to peak flow matching of a specific flow rate (or design storm), but more broadly requires the comparison of naturally occurring and post-project "flow rates and durations" over a range of flows (OC Watersheds 2010). The concept of flow duration matching is to incorporate hydromodification control BMPs such that the runoff flows and their durations do not differ from the baseline case over a specified range of flows. Plots showing flow versus duration are referred to as "flow duration curves[2]". The goal of the IHC is to integrate hydrologic controls into a proposed project such that the flow duration curve corresponding to the post-project (proposed) condition agrees with the baseline (natural) condition curve over the range of flows of interest. When this is accomplished, runoff from the proposed development will not theoretically increase erosive forces in the receiving stream channel.

3. ANALYSIS

A System Based approach was used to size hydromodification control BMPs within the Portola Center project. This approach models the actual drainage system and BMPs tributary to the points of compliance, namely where stormwater runoff discharges from

[2] A flow duration curve is a plot of flow rate (y-axis) vs. the cumulative duration, or percentage of time, that a flow rate is exceeded in the simulation record (x-axis).

the site. The System Based approach allows for a more complex BMP configuration and hydraulic outlet structure than the Unit Based Nomograph approach, which is what the Interim Sizing Tool was based on. Previously a site specific Unit Based Nomograph was created for the Portola Center site, however, given the changes in pre- to post-development drainage delineation, it was determined that the System Based approach was the more appropriate method of analysis to demonstrate compliance with the IHC. The project's Unit Based Nomograph was utilized to generally locate BMPs and provide initial sizing, but a System Based model, described herein, was subsequently implemented to fine tune the sizes and configurations of the BMPs.

The basic steps used to perform the System Based analysis include:

1. characterize site specific hydrologic conditions,
2. locate structural BMPs and select type,
3. select hydrologic modeling parameters,
4. define the flow range of interest,
5. select configuration of structural BMPs,
6. iteratively size BMP footprints to meet the IHC,
7. iterate BMP location and type (step 2), configuration (step 5), and size (step 6) to best meet proposed layout

The analysis performed is consistent with the technical memorandum titled *Assistance in Implementation of the South Orange County Hydromodification Standard: Alternatives to the Interim Sizing Tool* (OC Watersheds 2011). The specific assumptions used and results obtained for each one of these steps as it applies to the Portola Center project are provided below in Sections 3.1 to 3.7.

3.1 Step 1: Characterize Site Specific Hydrologic Conditions

The first step is to characterize the pre-development (natural) and post-development (proposed) hydrologic conditions in order to qualitatively understand the land use changes associated with the project. This characterization also forms a basis for selecting values of input parameters used for hydrologic modeling (Step 3). Hydrologic conditions, including drainage catchments, soil types, vegetation cover, impervious cover, and overland slope are discussed below and are summarized in Table 1.

3.1.1 Soil Type

Geologically, the site is primarily underlain with siltstone and sandstone of the Puente Formation – Soquel Member (Geocon 2011a, 2011b). The hydrologic soil groups assumed for the site include Type C and D soils. The Orange County Hydrology Manual Soils Map (OCEMA 1986) indicates that Type B soils were present on the site; however, site specific boring logs and the analysis of the geotechnical engineer indicate that this unit soil has properties more similar to a Type C or even Type D soil[3] (Geocon 2011a, 2011b, 2011c). Accordingly, the area with Type B designation per the Soils Map has been reassigned a classification of Type C soils for the runoff analysis. Exhibit 2 shows the change in characterized Hydrologic Soil Group based on site-specific geotechnical information. The same Hydrologic Soil Group delineation was used for both pre- and post-development.

3.1.2 Drainage Catchments

The Portola Center site generally drains from north to south. Due to the Project's location on natural ridge topography, runoff from the site drains to multiple channels tributary to Aliso Creek. Drainage delineations have been extensively altered from the natural condition due to previous grading operations onsite, performed in the 1990s[4]. While there were nine total discharge points from the site in the natural condition (pre-grading activities), only four primary outfall locations (A, B, C, and D) will receive runoff from new impervious surfaces in the post-development condition. The drainage delineations found onsite today ("existing condition") are more consistent with the

---

[3] The main source of Type B soils mapped on-site by the OC Soils Map is that classified as 113-Balcom Clay Loam. In looking at the geotechnical investigation performed by Geocon (2011a, 2011b), it appears that the native soils associated with this material are characterized as Puente Formation-La Vida Member (Tplv). According to this report, Tplv has "medium" to "high" expansion characteristics and the boring logs indicate this material is sandy to clayey siltstone with USCS classification as ML (low plasticity silt). Geocon was contacted specifically about the hydrologic soil group characterization for the Balcom Clay Loam. They stated that this "formational material should not be considered a type B soil because the infiltration rate would be very slow due to the cementation/density of the soil and the fines content. Based on the classifications, Geocon would expect the siltstones to be a Type C or D" (Geocon 2011c). Also the site specific geologic cross-section provided does not indicate a distinct change in soil material, in contrast to the regional map which shows a boundary between Type B and C soils.

[4] The existing Portola Hills development, north of the site, has also affected drainage areas running onto the site.

eventual post-development grading condition of the site. A preliminary assessment indicates that existing channels receiving runoff from the project site in its existing condition appear relatively stable (given the dense vegetation present within the channels), and have not experienced excessive geomorphic instability due to the alteration of the drainage area[5]. Given the stable condition of the existing channels receiving runoff from the site today and that the existing drainage delineations are more consistent with the eventual post-development grading, the existing condition was used as a primary basis for the pre-development drainage catchment delineation, shown on Exhibit 3[6]. A portion of northern offsite area, which formerly flowed to Outfall B in the natural condition, has been included in the pre-development catchment delineation because it is assumed the receiving stream of Outfall B evolved to handle this area prior to its diversion. The hydromodification analysis footprint shown on Exhibit 3 excludes portions associated with the existing Glenn Ranch Road and Saddleback Ranch Road because impervious cover will not be added to these areas as part of the development plan and these areas do not drain to proposed hydromodification BMPs.

The post-development drainage map for the site is provided in Exhibit 4. In Exhibit 4 the hydromodification analysis footprint has been delineated into subcatchments tributary to the four primary outfalls (A, B, C, and D). These outfalls also serve as the four points of compliance. Subcatchments that contain new impervious cover are routed into hydromodification BMPs whereas minor catchments that only contain landscaping are not routed through hydromodification BMPs.

3.1.3 Vegetation Cover

Natural vegetation cover is assumed to consist of grassland, shrubs, and chaparral based on historical aerial photographs. For the post-development condition, it is assumed that planting palettes for landscaped areas would consist primarily of native vegetation. Therefore, similar depression storage and overland roughness parameters were used for the pre- and post-development conditions.

---

[5] Preliminary assessment included review of historical aerial photo and field photos taken by Hunsaker and Associates in October 2011. Erosion was observed within a previously constructed Portola Hills Retarding Basin, however this basin, which receives off-site runoff, will be replaced in-kind as part of this project (BMP 2). Field photographs can be provided upon request.

[6] Although the drainage catchments for the pre-development condition are primarily based on the existing condition, the hydrologic parameters are based on the natural condition.

3.1.4 Impervious Cover

The pre-development condition was modeled as 99% pervious, associated with open space. In the post-development condition, impervious cover will include rooftops, asphalt pavement, and concrete walkway, which make up approximately 44% of the post-development footprint analyzed. This impervious cover is assumed to be directly connected to the stormdrain system. Impervious cover assumptions were based on the land use table in the Orange County Hydrology Manual (OCEMA 1986), except that landscaped areas were assumed to have 1% impervious cover instead of 15%, which is the assumption for flood control analyses[7]. Open detention basins were modeled as 100% impervious because rain that falls on the basin footprint contributes directly to runoff.

3.1.5 Overland Slope

Based on historical topographic maps (USGS 1982), the average overland slope of the pre-development (natural) condition is approximately 20%. The overland slope generally decreases in the post-development condition (ranges between 1% and 32%) because the site will be graded into flatter pads for development.

3.2 Step 2: Locate Structural BMPs and Select Type

Structural BMPs were located consistent with available space and the size requirements of the BMPs being analyzed. New impervious areas in the post-development condition were routed to at least one BMP location. Catchment delineations were made such that each BMP location has at least one tributary sub-catchment. The process of locating BMPs was an iterative process as site layouts changed during the planning process. The ten proposed BMP locations are shown in Exhibit 4 and the areas tributary to each are summarized in Table 2. All of the proposed BMPs are located in the southern portion of the project (Tentative Tract 15353). This is due to the fact that the site generally drains from north to south and so centralized BMPs in the southern tract maximize the area tributary to them. BMPs 1 through 4 discharge to Outfall A, BMPs 5 through 7 to Outfall B, BMPs 8a and 8b to Outfall C, and BMP 9 to Outfall D. While BMPs 2[8] and

---

[7] 15% imperviousness for landscaped areas is believed to be overly conservative for the purposes of the hydromodification analysis.

[8] BMP 2 is an in-kind replacement of an existing detention basin which was designed to meet peak flow matching requirements for runoff from the existing Portola Hills development. This system's stormdrain

6[9] are flow-by basins designed to meet flood control requirements, the others are flow-through hydromodification BMPs sized to meet the IHC at the points of compliance (Outfall A, B, C, and D). In terms of BMP type, BMP 9 is an open biotreatment detention facility while the other BMPs are rectangular underground vaults. It is assumed that pre-treatment will be provided prior to stormwater entering the hydromodification BMPs (e.g. silt/debris baffle boxes)[10].

3.3 Step 3: Select Hydrologic Modeling Parameters

Continuous hydrologic simulations were conducted to construct a continuous record of pre- and post-development runoff conditions, from which flow-duration curves were developed. For this project, continuous hydrologic simulations were performed with the USEPA's Surface Water Management Model (SWMM), version 5.0. The SWMM network modeled for the pre- and post-development conditions is provided on Figures 1a to 1d, respectively. All sub-catchment model parameters are listed in Tables 3, 4, and 5. The following information provides justification for specific parameters in these tables:

- Precipitation Data: The Trabuco precipitation record was used because it is the closest precipitation station to the project site with required long-term hourly precipitation data.
- Sub-Catchment Width: SWMM simulates subcatchment runoff as overland flow over a given width. The assumed pre-development catchment widths were calculated by dividing the sub-catchment area by an assumed natural flow path length of 350-feet, which is the approximate average overland flow path length

---

infrastructure eventually discharges at Outfall A, but will be kept separate from the Portola Center system and, thus, is not included in this hydromodification analysis.

[9] BMP 6 is needed to meet the peak flow matching criteria for flood control at Outfall B, but not for hydromodification management. BMP 6 will only store runoff during extremely high magnitude storm events. Although BMP 6 was included in the hydromodification modeling, this BMP is for flood control purposes and was not sized for hydromodification management.

[10] A high flow diversion associated with pre-treatment for each hydromodification BMP was modeled, however, the simulated runoff flowrates entering the hydromodification BMPs did not exceed the cutoff diversion rate. In other words, all simulated post-development runoff generated tributary to a hydromod BMP was routed through the BMP.

in the natural condition based on Geosyntec's evaluation of available contour maps of the area. The assumed post-development catchment widths were calculated by dividing the sub-catchment area by an assumed proposed flowpath length of 250-feet, which is a typical flow path length to each catch basin drop inlet.

- Slope: The pre-development slope was evaluated as the average overland slope on the site in the natural condition according to a USGS topo map. The proposed slopes of each subcatchment were calculated as an average of the overland slopes shown on the proposed grading plan for the Portola Center.
- Infiltration Parameters: The assumed pre-development hydraulic conductivity and Green-Ampt parameters for Type C and D soils is based on typical values, as referenced in *SWMM Hydrology: Runoff and Service Modules* (James et al, 2002). Infiltration parameters were also checked by comparing resulting runoff coefficients in SWMM with published runoff coefficients for soil type and slope range in *Hydrologic Analysis and Design* (McCuen, 2005). No reduction in hydraulic conductivity was assumed from pre- to post-development because the steeper slopes associated with pre-development were assumed to compensate for any reductions due to disturbance and compaction of fill material.

3.4 Step 4: Define the Flow Range of Interest ($0.1Q_2$, Q_5, and Q_{10})

In order to establish the flow ranges specified in the IHC, the pre-development 2-year (Q_2), 5-year (Q_5), and 10-year (Q_{10}) return period discharges were calculated for each outfall by constructing a partial-duration series from the pre-development simulation output as follows. The entire runoff time series generated by the pre-development simulation was divided into a set of discrete events. Discrete flow events were separated within the flow record when the flow rate dropped below a threshold value of 0.002 cfs/acre for a period of at least 24 hours. This partial duration series analysis is consistent with past technical guidance documents prepared for meeting the IHC (OC Watersheds 2010, 2011). The peak flow was determined for each event and ranked to establish the Q_2, Q_5, and Q_{10}, which are shown in Table 6. Consistent with the IHC, the low bound discharge is 10 percent of the Q_2 ($0.1Q_2$).

3.5 Step 5: Select Configuration of Structural BMPs

The geometry, outlet configuration, and infiltration rate were selected for each BMP so that it could be modeled as a storage unit in SWMM with a specific stage-discharge, stage-infiltration, and stage-storage relationship.

3.5.1 Hydraulic Outlet Configuration

The hydraulic outlet configuration dictates the stage-discharge relationship entered into the post-development scenario SWMM models. All hydromodification BMPs were modeled with a circular low flow orifice at the bottom and an overflow weir at the top. BMP 9 has two overflow weirs, one associated with the top of a riser pipe and another associated with a spillway overflow. An intermediate rectangular orifice was included for BMPs 3, 4, 5, 7, 8a, and 8b, which were added to most efficiently size the BMPs to meet the IHC. While the low flow orifices were sized to discharge below the approximate pre-development condition $0.1Q_2$ at the outfall point of compliance, the overflow weir was modeled to have a weir crest long enough to convey the design peak flowrate. Overflow weir dimensions were provided by Hunsaker and Associates. The proposed hydraulic outlet configuration for the proposed BMPs is provided in Table 7.

3.5.2 Geometric Configuration

The geometric configuration dictates the stage-storage or stage-area relationship entered into the proposed scenario model for each BMP. For BMP 9, the open detention basin, the stage-area relationship was provided by Hunsaker and Associates. The geometric configurations of the other BMPs modeled assumed a rectangular underground vault with vertical sidewalls.

3.5.3 Infiltration Rate

No infiltration was assumed for the BMPs due to the low permeability of soils in the post-development condition and formal direction from the City of Lake Forest Public Works Department that infiltration would not be permitted due to concerns that water infiltration into the fine grained soil conditions found onsite could cause soil heave and slope stability issues. The City's official determination was corroborated by the analysis and recommendations of the project geologist as well as the City's third party technical reviewers.

3.6 Step 6: Iteratively Size the BMP Footprints to Meet the IHC

Once the basic BMP configurations were established, the footprints of the underground detention BMPs were iteratively adjusted such that the simulated BMP discharge record met the IHC with a minimum amount of storage. The resulting BMP storage volumes for the proposed condition are summarized in Table 8. A graph showing the modeled distribution of storage area and cumulative volume for BMP 9[11], the open detention basin, is provided in Figure 2. The area and cumulative volume distributions are simple linear relationships for the other rectangular underground vaults since the area is constant from the bottom of the BMP to the top.

Figures 3a to 3d provide the flow duration curve comparison at each outfall to demonstrate that the post-development flow duration curves, with mitigation, are below the pre-development flow duration curve between $0.1Q_2$ and Q_5. Additionally, a peak flow frequency comparison is provided for each outfall in Figures 4a to 4d to demonstrate that between Q_5 and Q_{10}, the proposed post-development peak flows do not exceed the pre-development peak flows for any frequency interval; The IHC allows post-development flows to exceed natural peak flows by up to 10% for a 1-year frequency interval.

3.7 Step 7: Iterate BMP Locations, Types, Configurations, and Sizes to Best Meet Proposed Layout

Developing the proposed BMP plan shown in Exhibit 4 required an iterative modeling and planning process. If it was determined that relocating BMPs was necessary to meet the IHC and/or more effectively meet the proposed layout than the previous iteration, then the project team changed where BMPs were situated and returned to step 2. If it was necessary to adjust the size of the BMPs, then adjustments were made to the BMP configurations, and the project team returned to step 5. To demonstrate that the post-development BMP plan presented herein can accommodate the modeled storage volumes, Exhibit 4 shows where the BMP facilities are situated in plan view.

4. CONCLUSION

The System Based Sizing analysis conducted for the Portola Center project demonstrates that the proposed structural BMP plan for the site is sufficient to meet the flow duration control criteria specified in the IHC.

---

[11] BMP 9 was sized to meet water quality and peak flow matching requirements by Hunsaker. Although BMP 9 meets the IHC, it was not iteratively sized to minimize its BMP footprint because it was assumed that other design considerations govern its sizing.

5. REFERENCES

California Regional Water Quality Control Board. 2009. Waste Discharge Requirements for Discharges of Runoff from the Municipal Separate Storm Sewer Systems (MS4s) Draining the Watershed of the County of Orange, The Incorporated Cities of Orange County, and The Orange County Flood Control District Within the San Diego Region. Order No. R9-2009-0002. NPDES No. CAS0108740.

Geocon Incorporated. 2011a. Geotechnical Investigation – Portola Center North Tentative Map 17300 Lake Forest, California. May.

Geocon Incorporated. 2011b. Geotechnical Investigation – Portola Center South Tentative Map 15353 Lake Forest, California. May.

Geocon Incorporated. 2011c. Email Correspondence, Topic: "Site Specific Hydrologic Soil Group Designation." September.

James W., Huber W.C., Pitt R.E., Dickinson R.E., James W.R.C. 2002. SWMM Hydrology: Runoff and Service Modules.

McCuen R.H. 2005. Hydrologic Analysis and Design. Table 7.9, pg. 378.

Orange County Environmental Management Agency. 1986. Orange County Hydrology Manual.

Orange County Watershed Division. 2010. Technical Guidance Document for the South Orange County Hydromodification Control BMP Sizing Tool.

Orange County Watershed Division. 2011. Assistance in Implementation of the South Orange County Hydromodification Standard: Alternatives to the Interim Sizing Tool. Technical Memorandum.

United States Geologic Survey. 1982. El Toro Quadrangle Topographic Map.

TABLES

Table 1. Summary of Pre- and Post-Development Hydrologic Conditions

Hydrologic Characteristic	Pre-Development	Post-Development
Drainage Catchments[12]	Outfall A: 54.1 acres Outfall B: 94.7 acres Outfall C: 13.4 acres Outfall D: 13.4 acres Total: 175.6 acres	Outfall A: 55.9 acres Outfall B: 94.7 acres Outfall C: 13.9 acres Outfall D: 14.6 acres Total: 179.1 acres
Soil Types	Outfall A: 2% C, 98% D Outfall B: 32% C, 68% D Outfall C: 16% C, 84% D Outfall D: 67% C, 33% D Total: 24% C and 76% D	Outfall A: 9% C, 91% D Outfall B: 34% C, 66% D Outfall C: 0% C, 100% D Outfall D: 73% C, 27% D Total: 27% C and 73% D
Vegetation Cover	Native grassland, shrub, & chaparral	Landscaped areas assumed to have similar hydrologic characteristics as the natural condition
Impervious Cover	1%	Outfall A: 49% Outfall B: 41% Outfall C: 36% Outfall D: 49% Total: 44% (directly connected)
Overland Slope	20%	1% to 32%

---

[12] The drainage catchment acreage is different between the pre- and post-development conditions because the proposed grading plan creates a change in drainage delineations. Overall more area drains to the channels associated with Outfalls A, B, C, and D in the post-development condition than the pre-development.

Table 2. BMP Tributary Areas

BMP ID	Design Criteria	Type	Tributary Sub-Catchments	TributaryAcres
1	Hydromod/WQ	rectangular underground vault	A5	1.1
2	Flood (flow by)	rectangular underground vault	offsite	--
3	Hydromod/WQ	rectangular underground vault	A1, A3	36.7
4	Hydromod/WQ	rectangular underground vault	A4	10.3
5	Hydromod/WQ	rectangular underground vault	B1, B2	73.8
6	Flood (flow by)	rectangular underground vault	B1, B2	73.8
7	Hydromod/WQ	rectangular underground vault	B3	9.2
8a	Hydromod/WQ	rectangular underground vault	C1	4.2
8b	Hydromod/WQ	rectangular underground vault	C2	6.0
9	Hydromod/WQ/Flood (flow thru)	open detention basin	D1, D2	12.5

Notes:

(1) Sub-catchments A6, A7, B4, B5, C3, and D3 drain directly to their respective outfalls and are not routed to a structural BMP.

(2) BMP 2 is not modeled as part of hydromodification analysis. BMP 6 was modeled, but does not affect the hydromodification BMP sizing because the basin is for flood control.

Table 3. Pre-Development Sub-Catchment Parameters

Catchment ID	Catchment Area	Impervious Cover	Slope	Soil Type C	Soil Type D
	Acres	%	%	Acres	Acres
A	54.11	1.0	20	1.06	53.05
B	94.71	1.0	20	29.98	64.73
C	13.40	1.0	20	2.20	11.20
D	13.40	1.0	20	9.00	4.40
Total	175.62	1.0	20	42.24	133.38

Table 4. Post-Development Sub-Catchment Parameters

Catchment ID	Receiving BMP ID	Catchment Area	Impervious Cover	Slope	Soil Type C	Soil Type D
		Acres	%	%	Acres	Acres
A1	3	25.30	38.7%	3.9	2.40	22.90
A3	3	11.40	73.7%	3.3	0.00	11.40
A4	4	10.30	80.3%	8.7	2.80	7.50
A5	1	1.10	90.0%	1.5	0.00	1.10
A6	--	4.90	1.0%	20.0	0.00	4.90
A7	--	2.90	1.0%	15.9	0.00	2.90
B1	5	58.00	43.0%	4.7	20.10	37.90
B2	5	15.80	52.3%	3.7	11.45	4.35
B3	7	9.20	56.0%	1.0	0.60	8.60
B4	--	3.70	1.0%	9.0	0.00	3.70
B5	--	8.00	1.0%	7.8	0.00	8.00
C1	8A	4.20	47.5%	4.5	0.00	4.20
C2	8B	6.00	50.0%	5.0	0.00	6.00
C3	--	3.70	1.0%	14.2	0.00	3.70
D1	9	12.00	55.2%	1.0	9.60	2.40
D2	9	0.50	100.0%	8.3	0.50	0.00
D3	--	2.06	1.0%	32.1	0.56	1.50
Total		179.06	43.7%		48.01	131.05

Table 5. SWMM Parameters

PARAMETER	UNIT	VALUE
Subcatchment SWMM Parameters		
Modeled Area	Acres	Basin Specific
Flow Path Length	Feet	350 (Pre-Development)
		250 (Post-Development)
Slope (Pre-Development)	%	20
Slope (Post-Development)	%	Basin Specific
Imperviousness (Pre-Development)	%	1
Imperviousness (Post-Development)	%	Basin Specific
N-Imperv	--	0.012
N-Perv	--	0.15
Dstore-Imperv	Inches	0.02
Dstore-Perv	Inches	0.1
%Zero-Imperv	%	25
Modeled Soil Distribution		Basin Specific
Infiltration	Method	Green Ampt
Suction Head	Inches	8 (C Soils)
		10 (D Soils)
Conductivity	in/hr	0.07 (C Soils, Pre- and Post-Development)
		0.04 (D Soils, Pre- and Post-Development)
Initial Deficit	Fraction	0.26 (C Soils)
		0.21 (D soils)
Climatology SWMM Parameters		
Precipitation Gage		Trabuco Canyon (COOP ID 048992)
Evaporation	inches/month	60% of CIMIS Zone 4 Values
Storage SWMM Parameters		
Invert Elevation	ft	0
Maximum Depth	ft	Facility Specific
Initial Depth	ft	0

PARAMETER	UNIT	VALUE
Storage Curve	Method	Functional/Tabular
Functional Curve Coefficient	sq-ft	Facility Specific
Functional Curve Exponent		0
Functional Curve Constant		0
Link SWMM Parameters		
Conduit Length	ft	400
Conduit Manning's N	unitless	0.01
Conduit Shape		DUMMY
Orifice Type		Side
Orifice Inlet Offset	ft	0
Orifice Height	ft	Facility Specific
Orifice Discharge Coefficient	unitless	0.6
Weir Type		Transverse
Weir Inlet Offset	ft	Facility Specific
Weir Height	ft	2
Weir Length	ft	Facility Specific
Weir Discharge Coefficient	unitless	3
Simulation Options		
Modeled Period of Record		10/01/1948-05/03/2008
Routing Method	--	Kinematic Wave
Reporting Time Step	Minutes	60
Dry Weather Time Step	Minutes	240
Wet Weather Time Step	Minutes	15
Routing Time Step	Seconds	60
Dynamic Wave Inertial Terms	--	Dampen
Define Supercritical Flow By	--	Both
Force Main Equation	--	Hazen-Williams
Variable Time Step Adjustment Factor	%	75

Table 6. Partial Duration Series Results for Pre-Development Simulation

Peak Event	Flowrate (cfs)			
	Outfall A	Outfall B	Outfall C	Outfall D
Q10	49.96	86.29	12.29	12.01
Q5	43.89	75.80	10.80	10.53
Q2	30.40	52.34	7.47	7.27

Table 7. Proposed BMP Hydraulic Outlet Configuration

BMP ID	Total Depth (feet)	Low Orifice Height (feet)	Low Orifice Diameter (inches)	Rectangular Orifice Invert Height (feet)	Rectangular Orifice Opening Width (feet)	Rectangular Orifice Opening Height (feet)	Overflow Weir Crest Height (feet)	Overflow Weir Crest Width (feet)
1	14	0	1	N/A	N/A	N/A	12.0	1.0
3	14	0	5 1/4	7.5	1.5	2.0	12.0	15.0
4	14	0	2 1/2	8.0	0.75	1.0	12.0	6.0
5	14	0	7 3/32	9.0	5.0	2.0	12.0	22.0
7	14	0	2 9/16	10.0	1.0	1.0	12.0	5.0
8a	14	0	1 1/2	7.5	0.25	0.5	12.0	2.0
8b	14	0	1 3/4	7.5	0.5	0.75	12.0	2.0
9	10	0	3	N/A	N/A	N/A	6.0, 8.0	9.42, 10.2

Table 8. Proposed BMP Storage Volume

BMP ID	Storage Description	Draw-down Time (hours)	Overflow Depth (feet)	Total Depth (feet)	Top Footprint Area (square feet)	Top Footprint Area (acres)	BMP Volume Below Overflow (acre-feet)	BMP Total Storage Volume (acre-feet)
1	rectangular underground vault	34	12	14	508	0.01	0.14	0.16
3	rectangular underground vault	18	12	14	8,650	0.20	2.38	2.78
4	rectangular underground vault	33	12	14	3,630	0.08	1.00	1.17
5	rectangular underground vault	23	12	14	19,750	0.45	5.44	6.35
7	rectangular underground vault	27	12	14	2,795	0.06	0.77	0.90
8a	rectangular underground vault	37	12	14	1,362	0.03	0.38	0.44
8b	rectangular underground vault	38	12	14	1,967	0.05	0.54	0.63
9	open detention basin	47	6	10	22,321	0.51	1.52	3.28
Total					60,983	1.40	12.17	15.70
Total Underground Detention					38,662	0.89	10.65	12.43