Solutions for Urban Flood Mitigation and Stormwater Management 1st Edition by Guo

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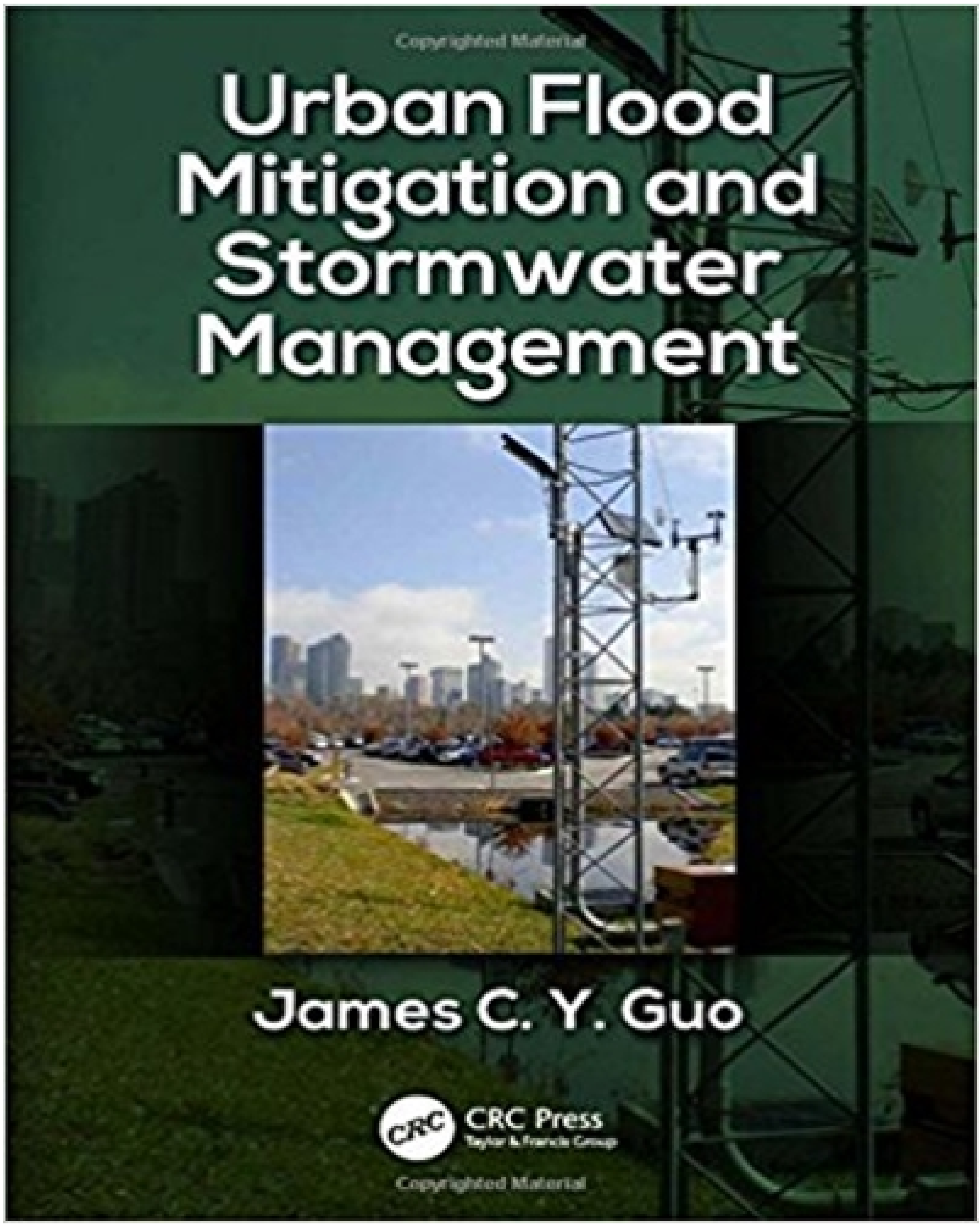

Solutions

STORMWATER MANAGEMENT

Low-Impact Development (LID) Design

Dr. James C.Y. Guo, P.E. Professor, Director,
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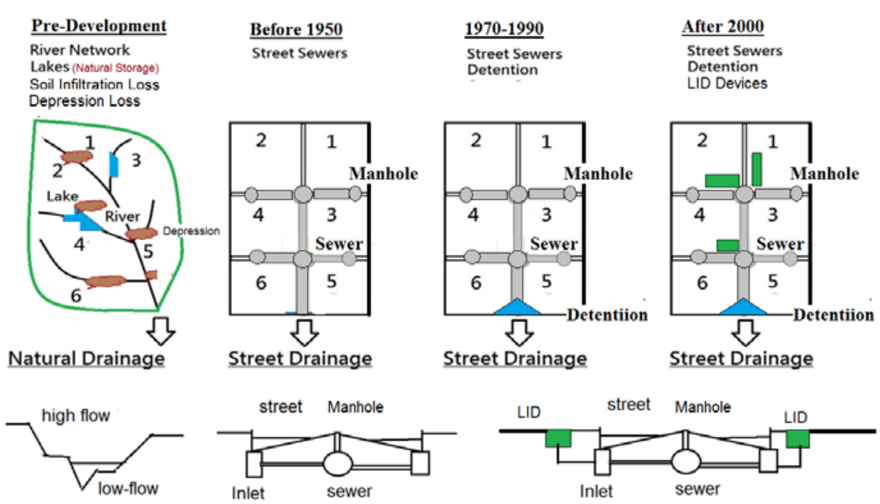
UC Denver

Natural Drainage Versus Urban Street Drainage

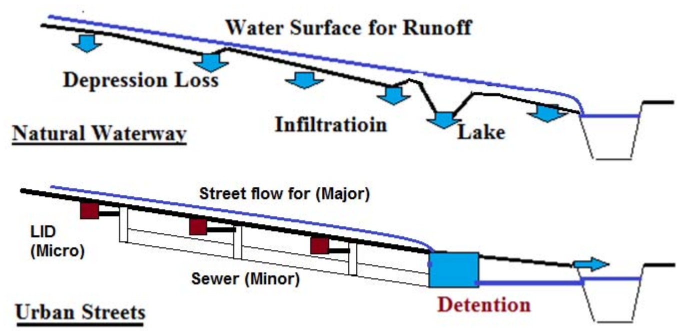


Hydro loss = Depression + Infiltration= WQCV + Detention

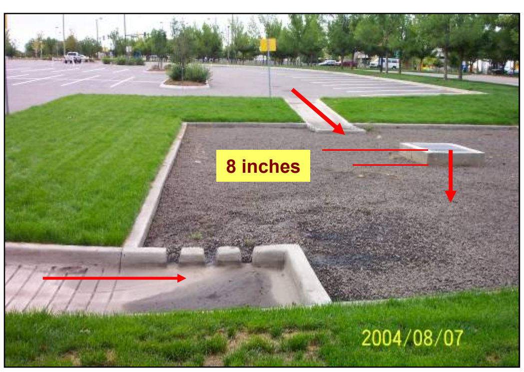
The major difference in watershed hydrology between the pre- and post-development is the amount of hydro losses. How to compensate the hydro loss after the development?

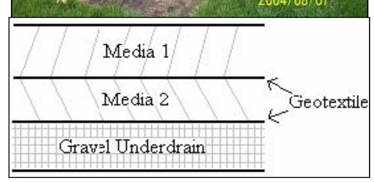
(1) On site LID storage volume = depression loss = 0.4 inch

(2) At the outfall point, the detention volume = infiltration loss = 1 to 2 inches



Challenges In Deriving WQCV



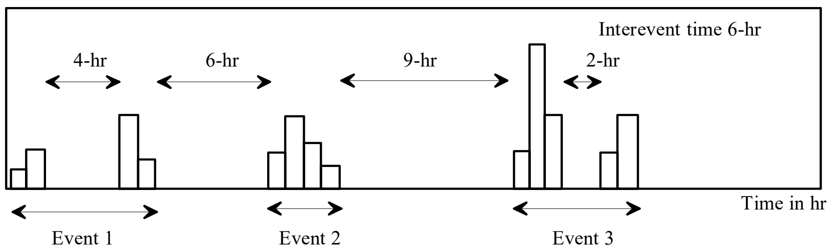


(Q-1) How to determine the runoff loading that should be infiltrated?
(Q-2) How to control flow release? Over a period of 6, 12, 24, or 40 hours?
(Q-3) How long is long enough for solids to settle?
(Q-4) How to design the overflow bypass?
(Q-5) How to determine the infiltration rate on the land surface
(Q-6) How to determine the seepage rate through the subsurface media?
(Q-7) How to cope with the clogging in the filtering media?

Inter-event time and LID's drain time

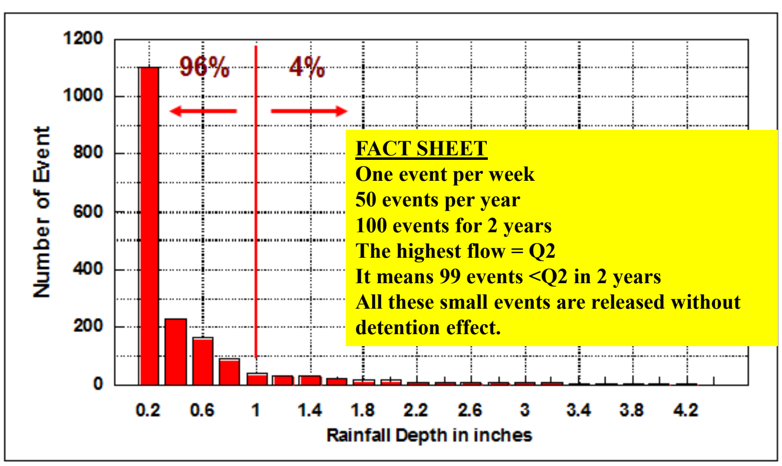
All rainfall records are continuous in time. How to separate a continuous record into single events for statistical analyses?

Example: Using a 6-hour inter-event time, how many single events are there in the following continuous rainfall record?
Setting LID's drain time to be 6 hours, what does it imply in the LID's operation?
Drain too fast ➔ WQ problems;
Drain too slow➔ overflow problems because the next event is coming.
Shall we set LID's drain time = average inter-event time?



FACTS (Guo **and Urbonas 1989, 1996)**

(a) 96% of rainfall population <1-hr 2-yr rainfall depth (1.0")
(b) only 4% of rainfall population are extreme events.
(c) average event-depth= 0.41 inch in Denver
(d) average yearly inter-event time = 110 hours in Denver
In the summer, the inter-event time = 12 hours



Extreme Event VS Frequent Event

- There are 100 events observed in a period of 2 years. The exceeding probability for the top one event out of 100 events is 1%. How do you compare this 1% of exceeding probability with the risk level of 1% for the 100-yr event.
- In fact, the top one in a period of 2 years is equivalent to a 2-yr event, or has a risk level of 50% for the 2-yr event.
- To avoid confusion, the conventional terms developed for extreme events shall not be used for WQ studies.
-

Extreme Event Study	Frequent Event Study
Non-exceeding probability	Runoff Volume Capture Rate
Detention volume in acre-ft Flow rate in cfs	WQ volume in inch/area Release rate in inch/hr
Precip-Duration-Freq (PDF)	Event depth
Time of Concentration in min	Drain time in hrs

Rainfall Statistics – Exponential Distribution for Event Depths

The runoff-producing rainfall depth is the difference between the recorded rainfall depth and the incipient runoff depth as:

$$p_i = P_i - I_s \quad (14.1)$$

in which p_i = runoff depth, P_i = rainfall depth, and I_s = incipient runoff depth. A value of *0.1* inch has been recommended as the incipient runoff depth (Discoll et al. in 1989).

$$P_a = \frac{1}{N}\sum_{i=1}^{i=N} P_i \quad (14.2)$$

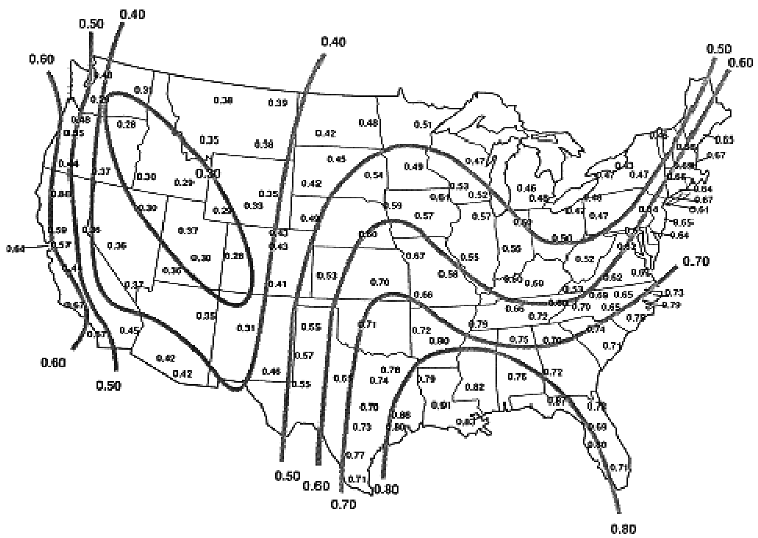
$$S_D = \frac{1}{(N-1)}\left[\sum_{i=1}^{i=N}(P_i - P_a)^2\right]^{\frac{1}{2}} \quad (14.3)$$

$$C_s = \frac{1}{S_D{}^3 N(N-1)(N-2)}\left[\sum_{i=1}^{i=N}(p_i - P_a)^3\right] \quad (14.4)$$

$$T_a = \frac{1}{N}\sum_{i=1}^{i=N} T_i \quad (14.5)$$

in which p_i = precipitation in the i-th event, P_a = average precipitation, N = total number of event in the record, S_D = standard deviation, C_s = skewness coefficient, T_i = time interval to the next event, and T_a = average interevent time.

EPA Field Investigation on Average Event Depth for the USA (MIT=6hr) For instance, Average Event Depth = 0.41 inch for Denver.



LID WQ Runoff Volume Capture Curves using Exponential Distribution

The exponential distribution is used to describe the rainfall depth distribution as:

$$f(P) = \frac{1}{P_m} e^{\frac{-P}{P_m}} \quad (1)$$

$$P_D(0 \le p \le P) = 1 - e^{\frac{-P}{P_m}} \quad (2)$$

Considering surface depression, the runoff-producing events produce the runoff volume as:

$$P_o = C(P - P_i) \quad (3)$$

in which P_o = WQCV in mm per watershed, *C*= runoff coefficient, P = design rainfall depth, and P_i = incipient runoff depth. Re-arranging Eq 1 yields:

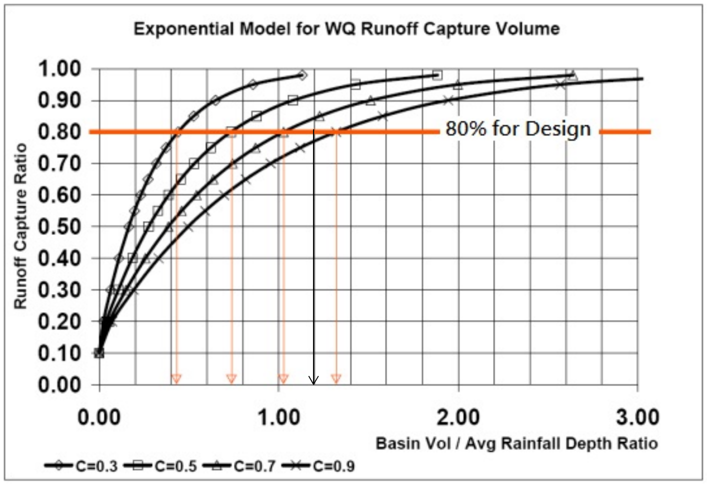
$$\frac{P}{P_m} = \frac{P_o}{CP_m} + \frac{P_i}{P_m} \quad (4)$$

$$P_D(0 \le p \le P_o) = P_D(0 \le p \le P) = 1 - e^{-(\frac{P_i}{P_m} + \frac{P_o}{CP_m})} \quad (5)$$

Re-arranging Eq 5 yields:

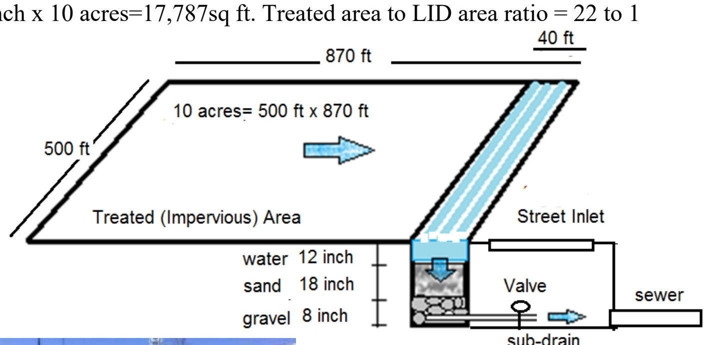
$C_v = 1 - ke^{\frac{-P_o}{CP_m}}$ and $k = e^{\frac{-P_i}{P_m}}$ where C_v= runoff capture ratio

Runoff Volume Capture Curves using Exponential Distribution

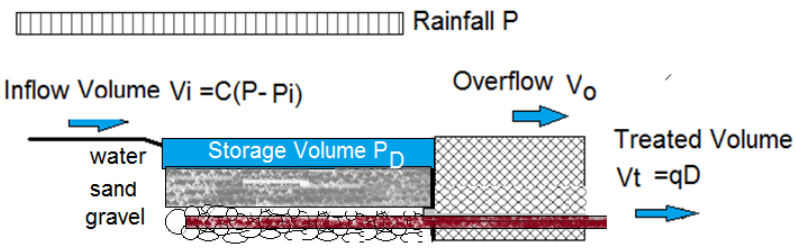


Design Example: US EPA suggests that the runoff capture ratio be 0.8 for LID/WQ designs
For a 10-acre tributary area with C=0.85 at Denver.
Avg Rainfall Depth=0.41 inch for Denver. WQCV=(1.2*0.41/12)*10=0.41 acre-ft

Design Example:
The average event rainfall depth=0.41 inch at Denver. US EPA suggests that the runoff capture ratio be 0.8 for LID designs. Treated area = 10 acres with C=0.85 at Denver.
WQCV=1.2*0.41=0.49 inch or WQCV= 1.2/12 *10 acre= 0.41 acre-ft
or WQCV= 0.49 inch x 10 acres=17,787sq ft. Treated area to LID area ratio = 22 to 1



Example for Runoff Volume Capture



An event is given with its rainfall depth and duration.

For the given event:

Inflow Volume Vi = C (P – Pi)
Treated Volume Vt = Flow-thru Volume=qD
Storage Volume Vs =P_D if (Vi – Vt)>P_D
Overtopping Volume Vo = (Vs- P_D) if Vo>0; otherwise Vo=0

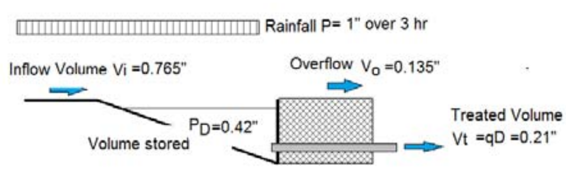
Example for Runoff Volume Capture

Set the storage volume P_D=0.42 inch/watershed with a drain time D= 6 hours. The runoff coefficient C=0.85. Determine the runoff volume capture for the event that has a total precipitation P=*1.0* inch in *3.0* hours.

$P_D = 0.42$ inch/watershed

The average release is:

$q = \frac{P_D}{D} = \frac{0.42}{6.0} = 0.07$ inch/hr



The basin's potential capture capacity is:

$Vi = C(P - P_i) = 0.85(1.0 - 0.1) = 0.765"$

$Vt = qT_d = 0.07 \times 3.0 = 0.21"$

$Vi - Vt = 0.767 - 0.21 = 0.557"$ >P_D

$Vs = 0.42"$ and *Vo=0.135"*

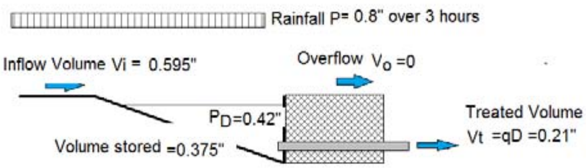
The runoff captured volume for this case = 0.63 inch per watershed
The runoff volume capture rate = (0.42+0.21)/0.765=82%

This is an event with an overflow.

Example of Runoff Event Capture

Set the storage volume P_D=0.42 inch/watershed with a drain time D= 6 hours. The runoff coefficient C=0.85. Determine the runoff volume capture for the event that has a total precipitation of P=0.8 inch in *3.0* hours.

$P_D = 0.42$ inch/watershed



The average release is:

$$q = \frac{P_D}{D} = \frac{0.42}{6.0} = 0.07 \text{ inch/hr}$$

The basin's potential capture capacity is:

$$Vi = C(P - P_i) = 0.85(0.8 - 0.1) = 0.595"$$

$$Vt = qT_d = 0.07 \times 3.0 = 0.21"$$

$$Vs = Vi - Vt = 0.595 - 0.21 = 0.374"$$

$$Vs = 0.374 < 0.452"$$

$$Vo = 0$$

The runoff captured volume for this case = 0.63 inch per watershed
The runoff volume capture rate = (0.374+0.21)/0.595=100%

This is an event with NO overflow.

Long term Cumulative Runoff Volume Capture

Basin StorageVolume	P	0.25	inch
Basin Drain Time	TD	6.00	hours
Catchment Runoff Coef	C	0.75	
Incipient Runoff Depth	Pi	0.10	inch
Average Release	q	0.042	inch/hr

Results	Toal Number of Event	Total Runoff Depth inch	Total Actural Cap Vol inch	Total Overflow Vol inch	Total Overflow Event
	95.000	25.777	18.650	7.127	16
Capture Rate		**0.723** for Volume		**0.832** for event	
Overflow Rate		**0.277** for Volume		**0.168** for Event	

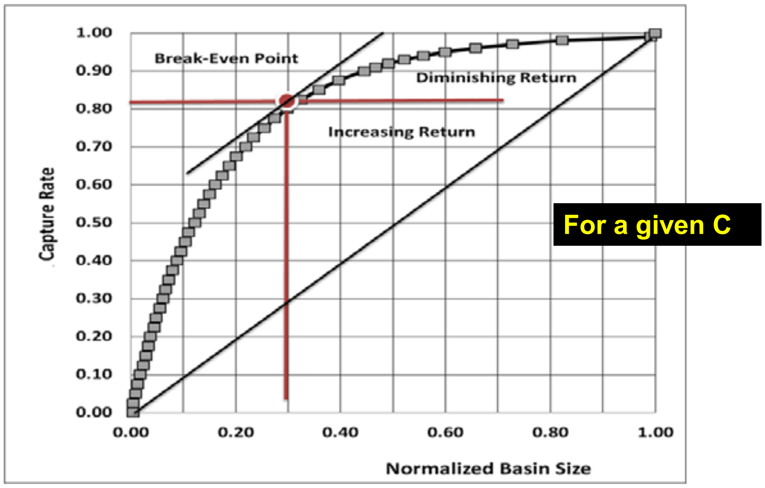
← **Summary**

No. of Event	Rainfall Depth inch	Rainfall Duration hour	Event Runoff Depth inch	Potential Capture Volume inch	Actual Capture Volume inch	Overflow Volume inch	Number of overflow event
1	0.120	2.000	0.015	0.333	0.015	0.000	0
2	0.130	1.000	0.023	0.292	0.023	0.000	0
3	0.400	2.000	0.225	0.333	0.225	0.000	0
4	0.120	2.000	0.015	0.333	0.015	0.000	0
5	0.280	1.000	0.135	0.292	0.135	0.000	0
6	0.180	1.000	0.060	0.292	0.060	0.000	0
7	0.250	9.000	0.113	0.625	0.113	0.000	0
8	0.850	3.500	0.562	0.396	0.396	0.167	1
9	0.210	9.000	0.083	0.625	0.083	0.000	1

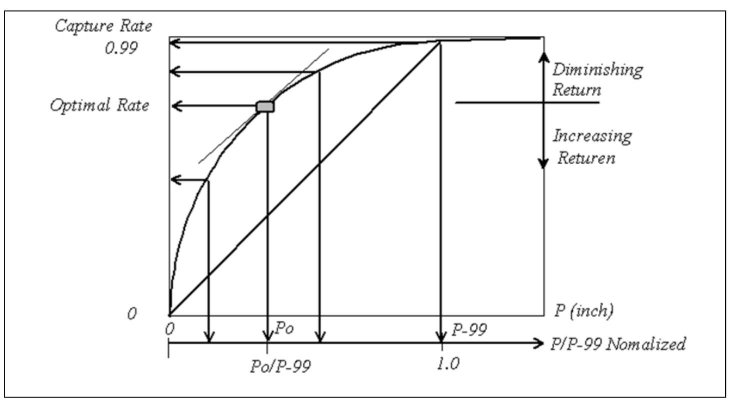
↓ **Data Continue**

Optimal Design- Diminishing Return

1. For a given runoff coefficient, C, let us set a range of strage volumes
2. For each storage volume, run 30 yr continuous rainfall record (1500 events) to find the overall runoff capture rate
3. Plot **capture rates** versus **storage volumes** for optimization



Development of Water Quality Control Volume



Optimization
(1) The bigger, the better
(2) Increasing return when small
(3) Diminishing return when big
(4) Average return =➔ optimal
(5) 80% runoff capture

Key Factors
(1) Location
(2) Watershed Imperviousness
(3) WQCB drain Time
(4) WQCB volume
(5) Runoff capture rate

Home Work: Develop Capture Volume Curve for Denver

inch	hour		inch	hour		inch	hour		inch
0.120	2.000	21	0.240	1.000	41	0.200	3.000	61	0.500
0.130	1.000	22	0.310	27.000	42	0.120	53.000	62	0.130
0.400	2.000	23	1.960	7.000	43	0.110	15.000	63	0.400
0.120	2.000	24	0.380	3.000	44	0.120	3.000	64	1.490
0.280	1.000	25	0.160	7.000	45	0.210	53.000	65	0.130
0.180	1.000	26	0.240	4.000	46	0.170	28.000	66	0.540
0.250	9.000	27	0.390	1.000	47	0.270	9.000	67	2.060
0.850	3.500	28	0.600	5.000	48	0.140	11.000	68	0.710
0.210	9.000	29	0.130	1.000	49	0.140	1.000	69	0.110
0.250	1.000	30	0.530	8.000	50	0.400	51.000	70	0.240
0.270	15.000	31	0.200	3.000	51	0.590	9.000	71	0.560
0.200	2.000	32	0.120	53.000	52	0.370	14.000	72	0.400
0.140	2.000	33	0.110	15.000	53	0.310	17.000	73	0.130
1.300	20.000	34	0.120	3.000	54	0.380	11.000	74	0.380
0.680	6.000	35	0.210	53.000	55	0.110	10.000	75	0.120
0.320	22.000	36	0.170	28.000	56	0.300	2.000	76	0.210
0.300	31.000	37	0.270	9.000	57	0.190	2.000	77	0.740
1.750	1.000	38	0.140	11.000	58	2.130	1.000	78	0.270
0.370	49.000	39	0.140	1.000	59	0.110	1.000	79	0.210
0.710	10.000	40	0.400	51.000	60	0.530	1.000	80	0.450

Derive WQCV for the cases specified.
Final values normalized by P-6 at Denver

Watershed	Basin	Drain	Time
Runoff	6 hr	12 hr	24 hr
Coeff	WQCV Normalized		
0.2			
0.4			
0.6			
0.8			

WQCV Regression Eqs

This method has been applied to the hourly continuous rainfall data recorded at *Seattle WA, Sacramento CA, Cincinnati OH, Boston MA, Phoenix AZ, Denver CO, and Tampa FL,* to find the optimal runoff capture volume for each of these sites. Findings from these seven gages form a data base for regression analyses using the model as:

$$\frac{P_o}{P_6} = aC + b$$

in which P_o = WQCV, P_6 = event average depth in EPA study, *a* and *b* = coefficients derived from regression analysis. For the seven gage sites, the regression equations show excellent correlation coefficients, r^2, ranging from *0.80* to *0.97*, depending on the drain time. Generally the equation for RECR has a higher correlation.

Drain Time	Volume Capture		Study	Event	Capture	Study
	a	*b*	*r-square*	*a*	*b*	*r-square*
12-hr	1.36	-0.034	0.80	1.1.96	0.010	0.97
24-hr	1.62	-0.027	0.93	1.256	0.030	0.91
48-hr	1.98	-0.021	0.84	1.457	0.063	0.85

Example on how to size WQCV basin

The tributary watershed of *2.0* acres is located in the City of Denver, Colorado. The watershed runoff coefficient is *0.41*. The storm water quality control basin will be operated with a drain time of 24 hours. Determine the WQCV.

For Denver area, a = 1.62 and *b = - 0.027* for the runoff volume capture. The WQCV to event average depth ratio is:

$$\frac{P_o}{P_6} = 1.62 \times 0.41 - 0.027 = 0.637$$

From the EPA study, the event average rainfall depth at the City of Denver is 0.41 inch. As a result, the WQCV is

$$P_o = 0.41 \times 0.637 = 0.26 \text{ inch/watershed}$$

Or, the storage volume is *0.043* acre-ft for an area of *2.0* acres.

What if the basin is located in Boston?

Denver Storm Water Quality Capture Volume (Depth) (WQCV)

The WQCV is calculated as a function of imperviousness and BMP drain time using Equation 3-1, and as shown in Figure 3-2:

$$WQCV = a(0.91I^3 - 1.19I^2 + 0.78I) \qquad \text{Equation 3-1}$$

Where:

WQCV = Water Quality Capture Volume (watershed inches)

a = Coefficient corresponding to WQCV drain time (Table 3-2)

I = Imperviousness (%/100) (see Figures 3-3 through 3-5 [single family land use] and /or the *Runoff* chapter of Volume 1[other typical land uses])

Table 3-2. Drain Time Coefficients for WQCV Calculations

Drain Time (hrs)	Coefficient, a
12 hours	0.8
24 hours	0.9
40 hours	1.0

Guo, James C. Y. Urbonas, B. and MacKenzie K. (2014) "Water Quality Capture Volume for LID and BMP Designs", ASCE J of Hydrologic Engineering, Vol 19, No 4, April, pp 682-686

Guo, James C.Y. and Urbonas, Ben. (2002). "Runoff Capture and Delivery Curves for Storm Water Quality Control Designs," ASCE J. of Water Resources Planning and Management, Vol 128, Vo. 3, May/June.

Guo, James C.Y. and Urbonas, Ben (1996). "*Maximized Detention Volume Determined by Runoff Capture Rate*," ASCE J. of Water Resources Planning and Management, Vol 122, No 1, Jan.

How to Size the WQCV out of the US Cointinent

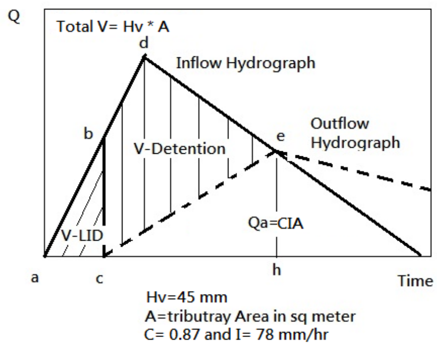
Taiwan's approach for LID and Detention Designs

On site LID and Detention Storage Volume = 45 mm per m^2

保育水量 (LID for WQ + Detention for Peak Reduction)

Infiltration amount	1.095	inch	27.82	mm		
Depression loss	0.600	inch	15.24	mm		
Interception loss	0.100	inch	2.54	mm		
Total	1.795	inch	45.60	mm	0.045599	m

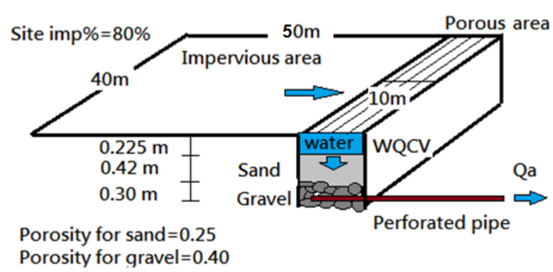
On site sewer capacity= CIA = 0.87*78mm/hr*1.0 m^2 = 0.000019 cms/m^2



How to relate these design criterion to Taiwan's rainfall patterns? How to select a proper drain time?

How to combine LID Designs with Detention Ponds for Flood Control?

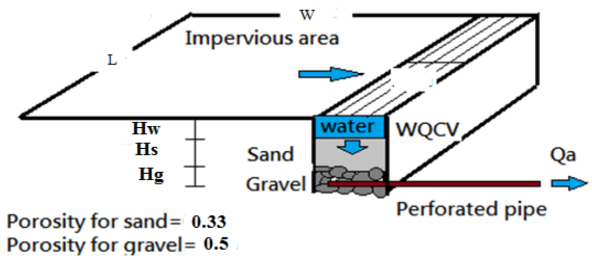
Design Example



Total area 40 x 50 = 2000 m^2
Required on-site storage volume: WQCV= 45 mm *2000 = 90 m^3
LID area = porous area = 10 *40=400 m^2
On-ground water storage depth = 90 m^3/400 m^2=0.225 m
Set the porosity for sand layer=0.25 and sand layer thickness= 0.42 m
Set the porosity for gravel layer=0.40 and gravel layer thickness= 0.30 m
Underground water storage depth= 0.25*0.42+0.30*0.40=0.225 m (o.k.)
Set the drain time = 12 hr
Infiltration rate = water storage depth/drain time=0.225 m/12 hr=18.75 mm/hr

How to find the sand-mix that sustains 18.75 mm/hr (0.75 inch/hr) ?!
Is this infiltration rate with or without clogging?
What is clogging in LID devices?

LID Example



Total area A= L x W= 200 x 150 = 30,000 ft^2
Required on-site storage volume: WQCV= 0.5 inch *30,000 ft ^{2}= 1250 ft^3
LID area = porous area = 50 *25=1250 ft^2
On-ground water storage depth Hw = 1250 ft^3/1250 ft^2=1.0 ft = 12 inch
Set the porosity for sand layer=0.33 and sand layer thickness Hs = 24 inch
Set the porosity for gravel layer=0.50 and gravel layer thickness Hg = 8 inch
Underground water storage depth= 0.33*24+0.50*8=12 inch
Set the drain time = 12 hr
Infiltration rate = water storage depth/drain time=12 inch/12 hr=1 inch/hr
How to design the sub-base to have an infiltration rate at 1 inch/hr !!!!

Source of Pollutants In Storm Water

Sediment Deposit
at Points of Flow Interception

Evidence of Pollutants in Storm Water

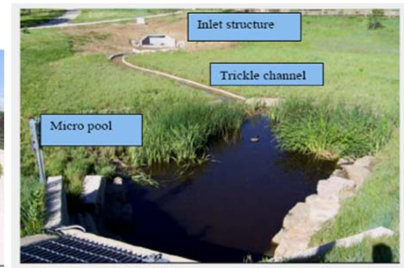
Street Sweeping
Frequent Rainfall Events
Release Control
Overflow Bypass

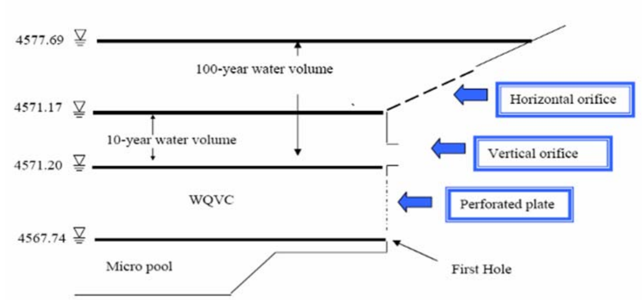
I-15 and Spring Mountain Rd, Las Vegas

WQ Infiltrating Basins and Ponds

UDFCD Field Test

Grant Ranch Basin Study





Removal of Metals by Micro Pool

		Units	Mean	Median	Standard deviation	Upper 95% CI	Lower 95% CI
Total recoverable							
Chloride	**Inflow**	mg/L	6.3	5.0	2.9	12.0	3.3
	Outflow	mg/L	14.4	10.5	2.2	32.9	6.3
Copper	**Inflow**	ug/L	12.5	15.0	1.9	19.5	8.0
	Outflow	ug/L	5.8	6.0	1.5	8.8	3.8
Magnesium	**Inflow**	mg/L	2.4	2.1	1.4	3.6	1.6
	Outflow	mg/L	3.5	3.4	1.1	4.9	2.6
Manganese	**Inflow**	μg/L	96	130	2	180	52
	Outflow	μg/L	54	40	2	200	12
Zinc	**Inflow**	μg/L	90	93	2	130	62
	Outflow	mg/L	36	40	1	56	23
Soluble							
Copper	**Inflow**	μg/L	6.2	5.0	1.6	11.0	3.5
	Outflow	μg/L	4.2	4.5	2.5	11.2	1.6
Zinc	**Inflow**	μg/L	15.1	14.8	1.9	30.2	7.5
	Outflow	μg/L	14.0	10.3	3.3	33.2	4.0

Q and A

STORMWATER WQCV for LID Designs

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Professor and Director,
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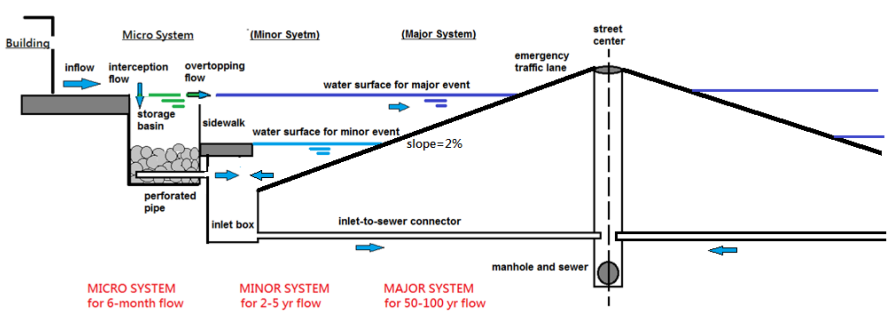
Colorado, USA

Examples of LID Landscape

Low-Impact Development Concept

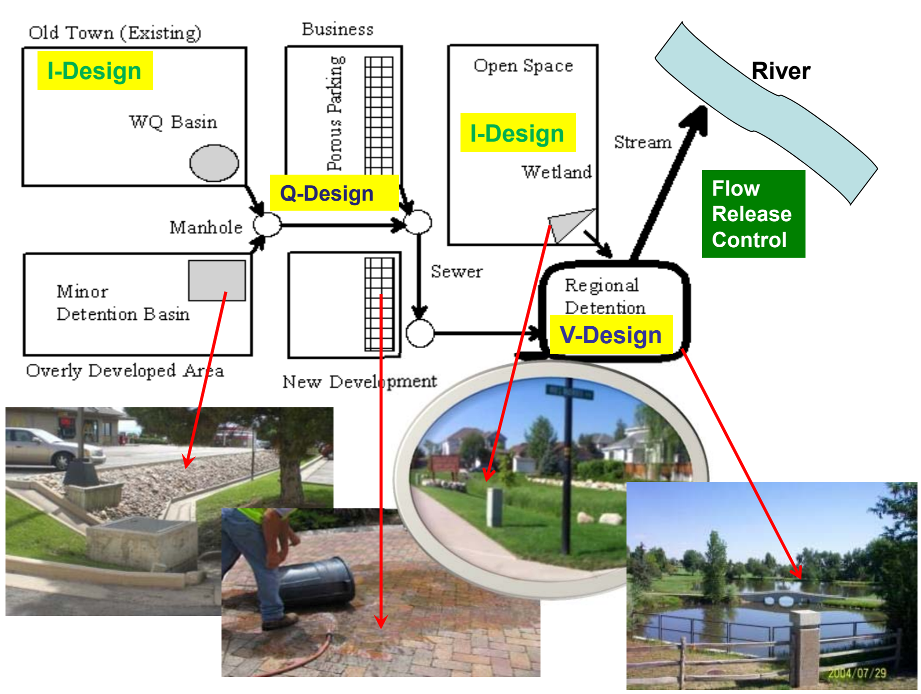
- Watershed development reduces soil infiltration and depression loss. LID is a concept to preserve the watershed pre-development regime.
- A LID device is designed to infiltrate storm runoff into “filtering” (過濾) layers
- A LID device is sized to cope with the WQ issues associated with frequent events or it can be expanded into an extended stormwater detention basin to manage the peak flow reduction in extreme events.
- How to incorporate LID devise into an existing urban drainage system=➔3 M cascading flow system



Urban Drainage=Micro + Minor + Major Systems

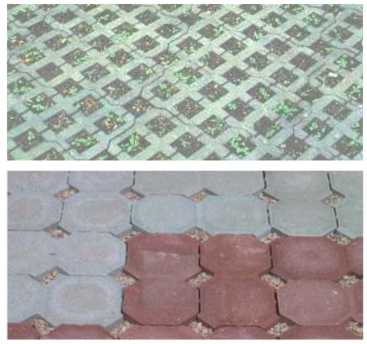
Purpose	Designs	Facilities
Filtering	LID **I-design** (micro)	infiltration, porous areas
Conveyance	Flow **Q-design** (minor/major)	streets, sewers, channels,
Storage	Storage **V-design** (major)	detention, retention, wetland areas

Note: Micro=6-month, Minor=2-yr, Major=100-yr



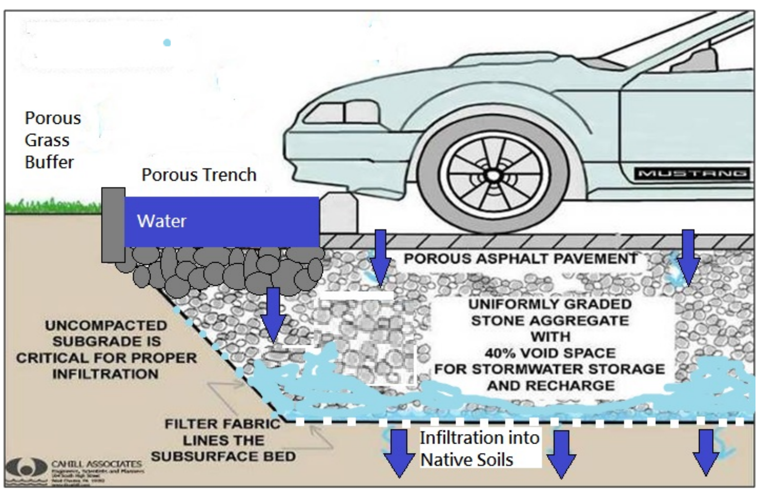
Structured Pavers for Water Infiltration

1. Grass Buffer
2. Grass Swale
3. Modular Block Porous Pavement
4. Cobble Block Porous Pavement
5. Porous Concrete Pavement
6. Porous Gravel Pavement
7. Porous Pavement Detention
8. Porous Landscape Detention
9. Sand Filter
10. Extended Detention Basin
11. Constructed Wetland Basin
12. Retention Basin



○Guo, James C.Y. (2010)"Preservation of Watershed Regime for Low Impact Development using (LID) Detention", ASCE J. of Engineering Hydrology, Vol 15, No 1., January, 2010
○Guo, James C.Y., Kocman, S and Ramaswami, A (2009) "*Design of Two-layered Porous Landscaping LID Basin,*", ASCE J. of Environ Engineering, Vol 145, Vol 12, December.

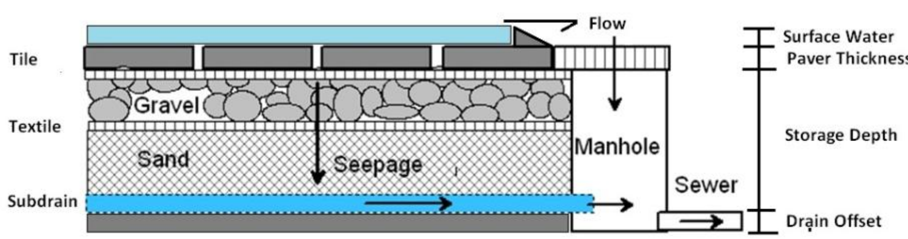
Native Soils for Water Infiltration



Grass Swale, Grass Buffer, Unpaved/Pervious Parking lots, Play Grounds, Picnic Park, Rain Gardens, Tree box, etc.

Porous Pavements for LID Settings – Source Control

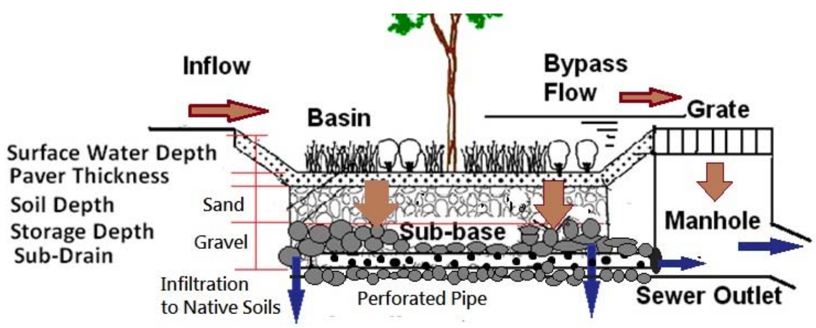
Porous Pavers for Stormwater Disposal --- Source Control



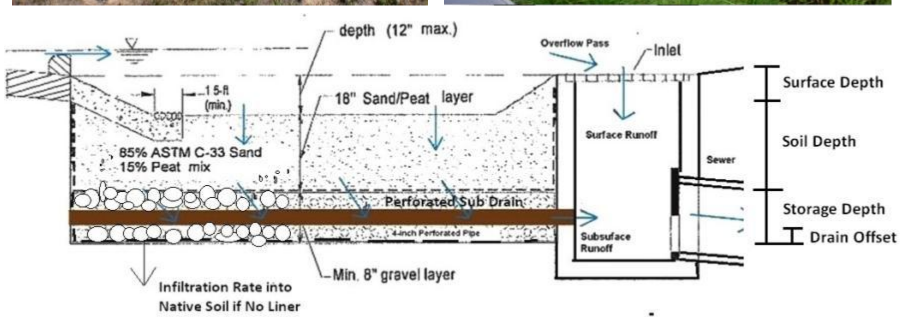
Any suggestion to improve these designs?

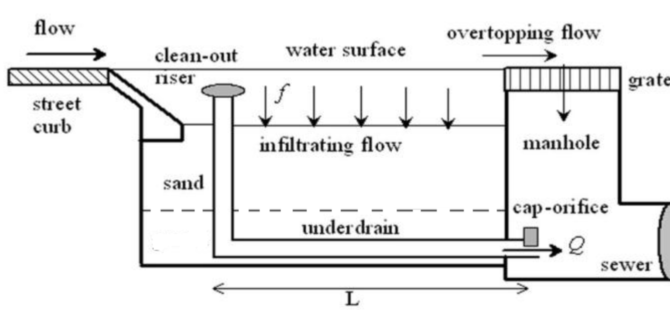
Landscape Porous Basin = Bio Retention Basin

Dimension for Infiltrating Basin -- ON SURFACE
1)Basin Storage Volume WQCV and Drain Time, T_d
2)Basin Depth, Y such as 12 in; Basin Area, A =WQCV/Y
3)Inflow Spreader and Bypass Flow Weir
Dimension for Sub-Base Structure -- Through SUBSURFACE
A.Top layer -- Sand mix
B.Bottom layer -- Gravel
C.Perforated Pipe



Bio Retention Basin Designed for 3-6 month Event





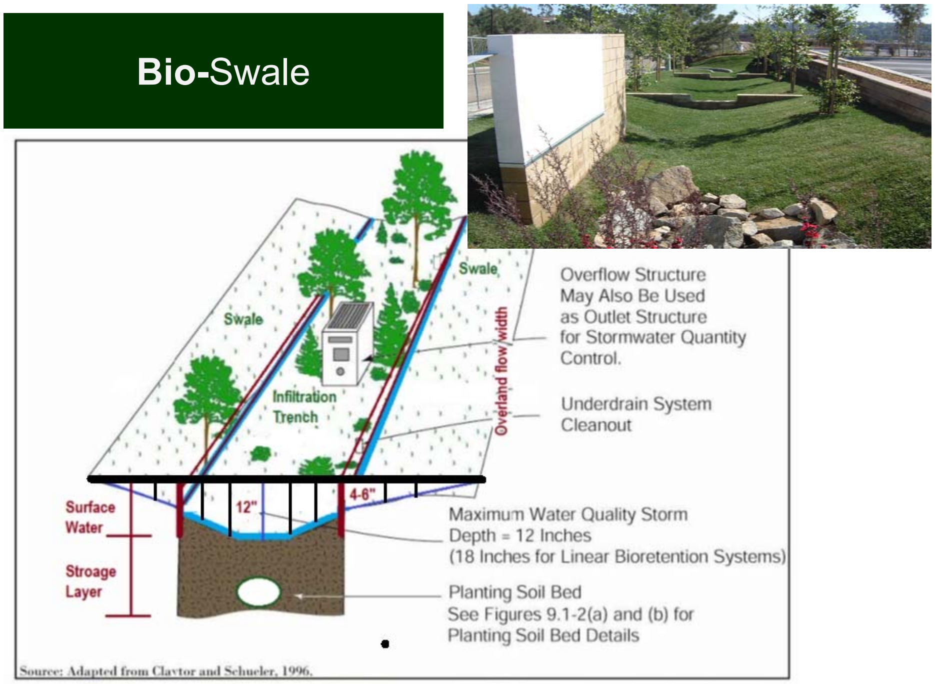
ExampleConstruction of RAIN GARDEN

New construction at a shopping mall area, Denver, Colorado

Urban Renewal Project
14th Street
Downtown Denver

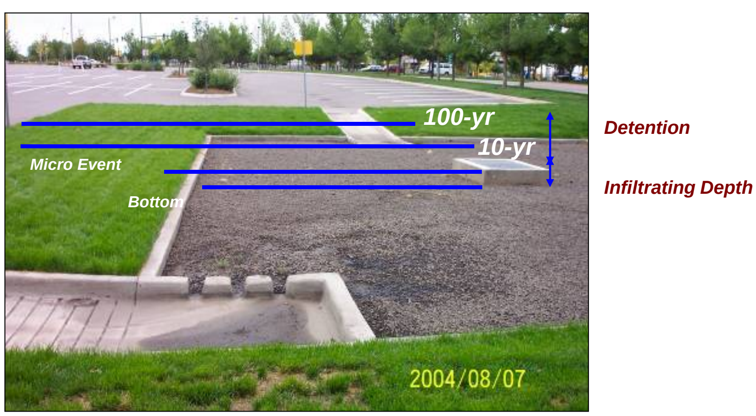
Tree boxes are used as a Stormwater Outlet into native soils

Bio-Swale



Infiltration Basin and Flood Detention

- Flood Control System for Minor + Major Events (10-yr and 100-yr floods)
  (1) Conveyance and (2) Detention Storage
- Stormwater Management System for Micro Events (3 to 6 month events)
  (3) Infiltrating and filtering

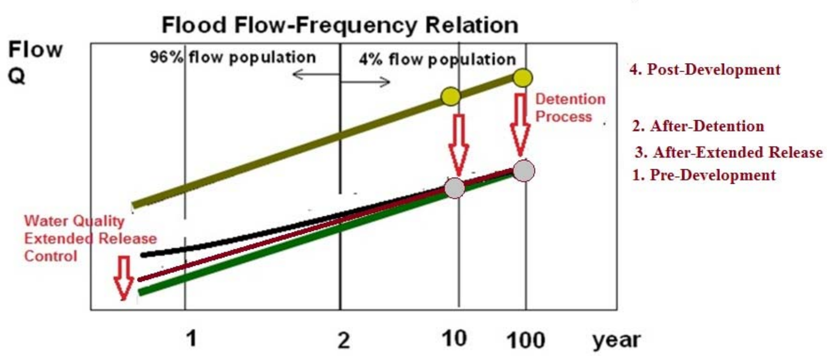


Evolution of Urban Drainage Design

Before 1970	1970-1990	1990-2000	2000-2010
Pass Q	Reduce Q 2- to 100-yr events	Reduce Q For All events	Reduce Q and V For All events
Flood Conveyance	Flood Conveyance + Flood DB Control	Flood Conveyance + Flood DB Control + SW BMP	Flood Conveyance + Flood DB Control + SW BMP + Watershed LID
Inlets, Sewers, Streets, Channels	Detention Basins Retention Basins	Retrofitted Outlet Control	Porous Pavers LID Watershed

○Green Concept =➔ Preservation of Natural Watershed or the flow-frequency curve remains unchanged. It implies that the LID layout should mimic the porous and cascading flow processes in the natural watershed.

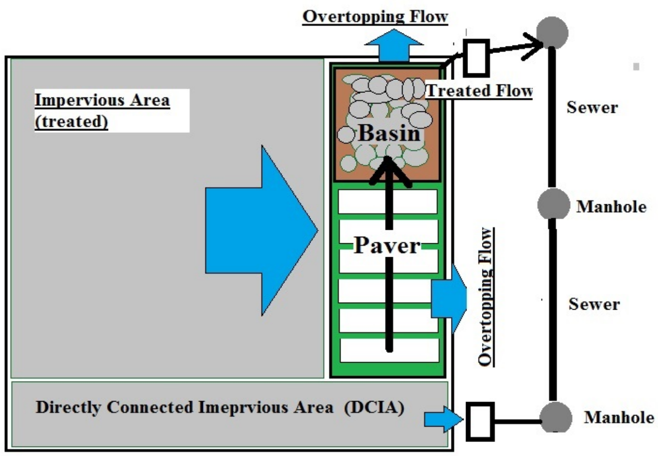
○GREEN = PRESERVATIONOF WATERSHED REGIME



- ○ Curve 1 = Pre-development flows (Natural System)
- ○ Curve 4 = Release transported by Conveyance System
- ○ Curve 3 = Release control using Detention to reduce Q-10 and Q-100
- ○ Curve 2 =Extended Release for WQ, LID, to reduce V

○Guo, James C.Y. (2010)"Preservation of Watershed Regime for Low Impact Development using Detention", ASCE J. of Engineering Hydrology, Vol 15, No 1., January.

Layout of LID Site – Small Lot (1-5 hectares)



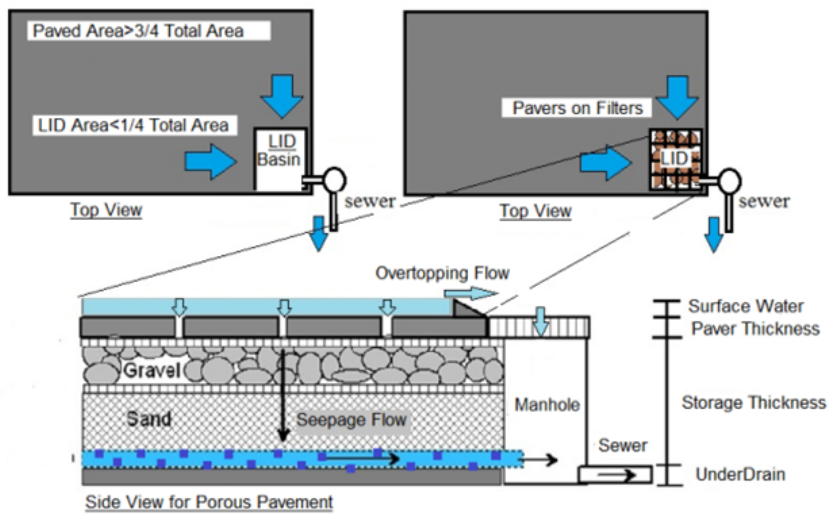
Total area is divided into Impervious area, LID area, and DCIA area
Impervious area drains onto the LID area, the area ratio>4.
Not all impervious area can be connected to a LID area

What is wrong with these LID sites?

Soft or Hard Surface?
Loading Strength?

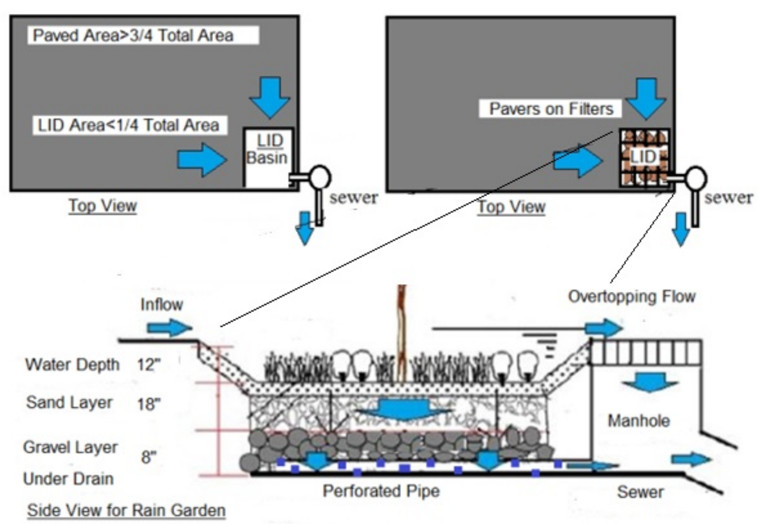
Flow-over LID = gravel reservoir into sand filter

Conveyance LID –Flow over
Porous Block Paver, Porous Concrete, Pervious Asphalt Surfaces



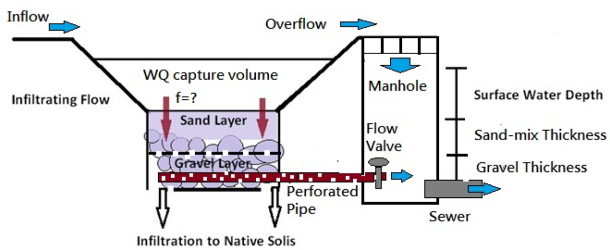
Flow-stored LID = surface reservoir into sand filter

Storage LID – Volume-stored
Rain Garden, Bio-Retention, Sand Filter , Infiltration Basin, and Rain Barrel



Basic Design Parameters and Considerations

Design Parameters
1. Drain Time
2. WQCV
3. Infiltration Rate
4. Sand-mix Layer
5. Gavel Layer
6. Flow Valve
7. Clogging Effect



Design Considerations
(Q-1) How often it rains?
(Q-2) How much runoff volume shall be stored? how big is big enough?
(Q-3) How fast to drain the stored water? how long is long enough for WQ?
(Q-4) How to control the flow release rate?
(Q-5) How to design the overflow bypass?
(Q-5) Is infiltration on the land surface = seepage rate through subsurface ?
(Q-6) How to evaluate the effectiveness?
(Q-7) How to assess the clogging effect?

Storm Water Quality Capture Volume (Depth) (WQCV)

The WQCV is calculated as a function of imperviousness and BMP drain time using Equation 3-1, and as shown in Figure 3-2:

$$\text{WQCV} = a(0.91I^3 - 1.19I^2 + 0.78I) \qquad \text{Equation 3-1}$$

Where:

WQCV = Water Quality Capture Volume (watershed inches)

a = Coefficient corresponding to WQCV drain time (Table 3-2)

I = Imperviousness (%/100) (see Figures 3-3 through 3-5 [single family land use] and /or the *Runoff* chapter of Volume 1[other typical land uses])

Table 3-2. Drain Time Coefficients for WQCV Calculations

Drain Time (hrs)	Coefficient, a
12 hours	0.8
24 hours	0.9
40 hours	1.0

Guo, James C. Y. Urbonas, B. and MacKenzie K. (2014) "Water Quality Capture Volume for LID and BMP Designs", ASCE J of Hydrologic Engineering, Vol 19, No 4, April, pp 682-686

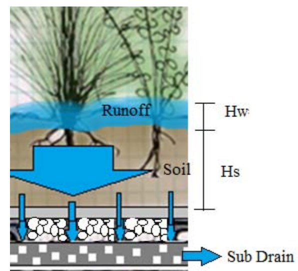
Guo, James C.Y. and Urbonas, Ben. (2002). "Runoff Capture and Delivery Curves for Storm Water Quality Control Designs," ASCE J. of Water Resources Planning and Management, Vol 128, Vo. 3, May/June.

Guo, James C.Y. and Urbonas, Ben (1996). "*Maximized Detention Volume Determined by Runoff Capture Rate*," ASCE J. of Water Resources Planning and Management, Vol 122, No 1, Jan.

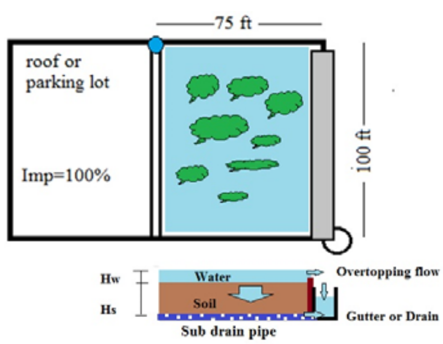
Example of Flat-Bed Green Roof Design
How thick the soil layer is (water absorption) ?

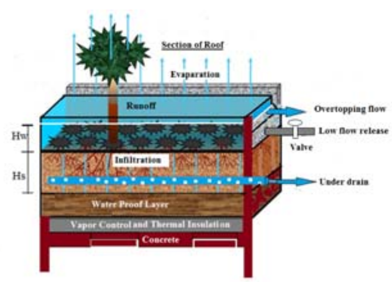
Top View

Vertical Profile



Flat-bed Green Roof -- Runoff Loading and Soil Thickness





Determine the depth of soil layer for the green roof of 75-ft by 100-ft

Set Hw= WQCV for I=100%. Set the drain time to be 24 hours.

With I=1, WQCV=0.9*(0.91*1.0^3-1.19*1.0^2+0.78*1)=0.44 inch for 24-hr drain

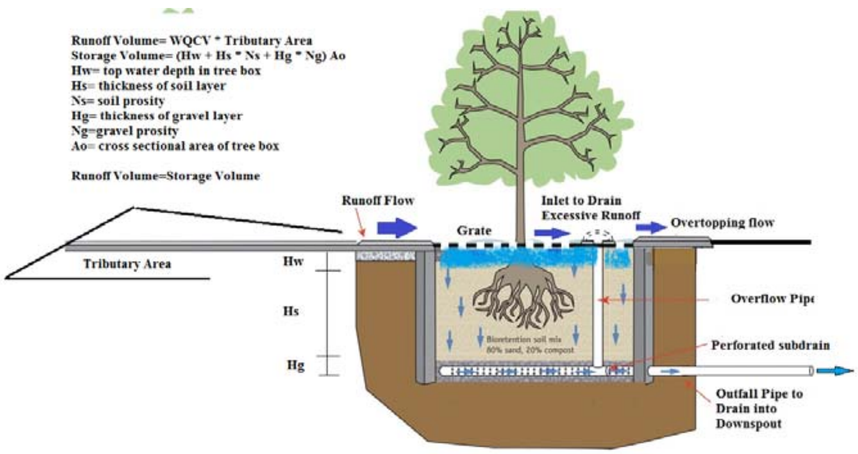
Stored Water Volume= Flat Bed Area * WQCV/12 = 75*100*0.44/12=275 ft^3

Soil Depth for Green Roof

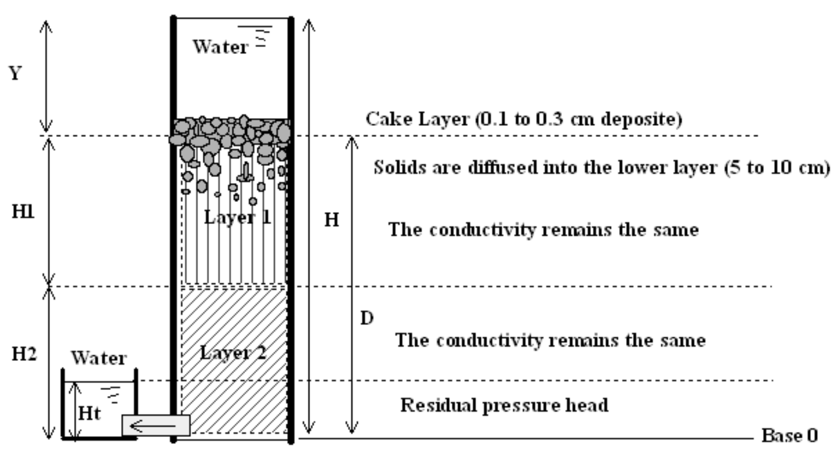
Set Hw= WQCV = 0.44 inch, Soil Porosity =0.25, Initial water content =0.10

Soil Depth = 0.44/(0.25-0.10) = 3.0 inch, use Hs= 4 to 6 inches due to roots and others

Storage-bed Green Roof-- Tree Box Design



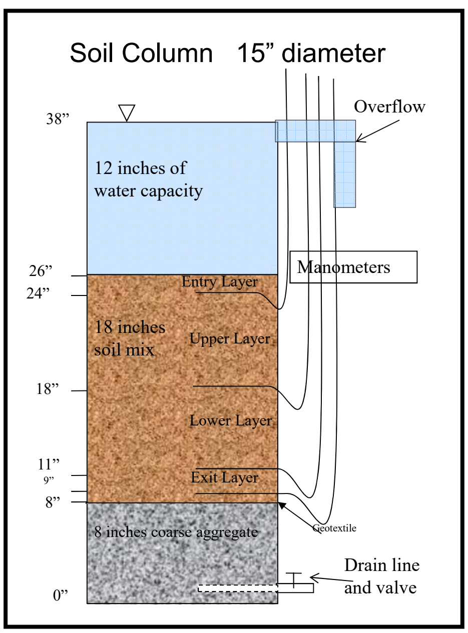
Clogging Condition -- Observed in Lab



Laboratory Tests on Soil Mix and Infiltration Rate

Infiltration Rate =fct (Mix of Sand, Soil, Others etc, and Clogging Effect)

INFILTROMETER DESIGN



Material for Sand mix

Material for Sand Mix	Density (pound/yard^3)	Cost (2009) ($/cubic yard)
Peat (Paulino Gardens)	700 lbs/cy	$130
A1 Compost (Pioneer)	1,030 lbs/cy	$35
Shredded paper (WM)	39 lbs/cy	Variable, but very cheap
Sand (Pioneer)	2,700 lbs/cy	$17
Rubber (Acugreen)	2,000 lbs/cy	$17
¾ aggregate (Pioneer)	2,800 lbs/cy	$25
Crushed concrete (Oxford Recycling)	2,900 lbs/cy	$11

Three Types of Soil Mix tested for Sub-base Medium

Soil mix is composed of sand, compost, paper, old tire particles, peat, crushed bottles, recycled concrete blocks etc. We like to know the infiltration rate, chemical leaching, clogging effect, and cost. After an extensive review of urban waste material, the following 3 mixes are developed and tested:

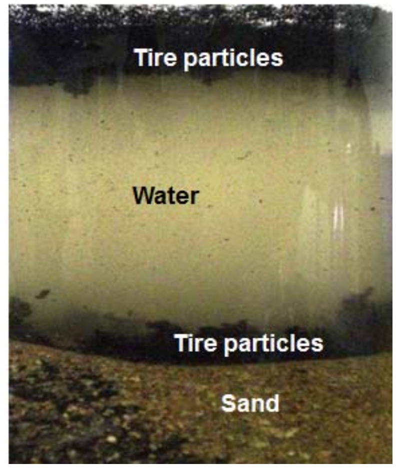
(1) Type 1 (Control) =15% peat and 85% sand

(2) Type 2 = 7.5%compost, 7.5% shredded paper, 85% sand

(3) Type 3 = 7.5% compost, 7.5% shredded paper, 8% tires, 77% sand

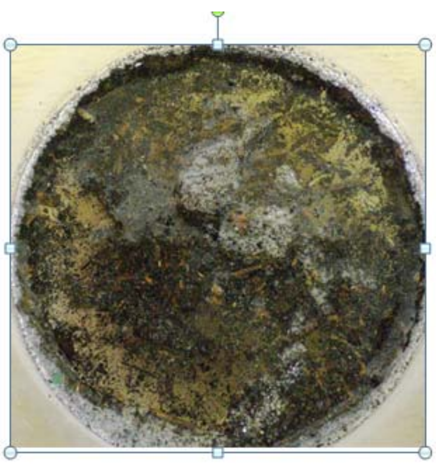
CHALLENGES:

Peat, Paper and Tire particles are floatable after the medium becomes saturated. How does “density stratification” affect the infiltration rate?



Sand and Tire Mix During First Flush

Sand and Tire Mix During 2nd Flush



Float on water surface

Sand+Tire+Compost+Paper Mix after Several Flush

Surface Deposit (Cake Layer) after Long-term Infiltration Simulation with Sediment-laden Stormwater

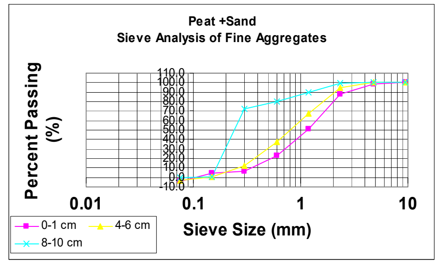
Sand and Peat Mix

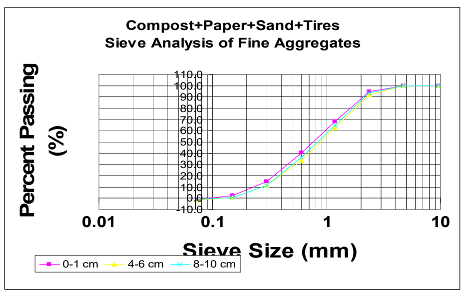
Surface Deposit (Cake Layer) after Long-term Infiltration Simulation with Sediment-laden Stormwater

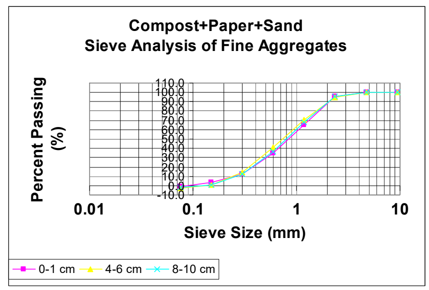
Sand + Compost + Paper Mix

GRAIN SIZE DISTRIBUTIONS

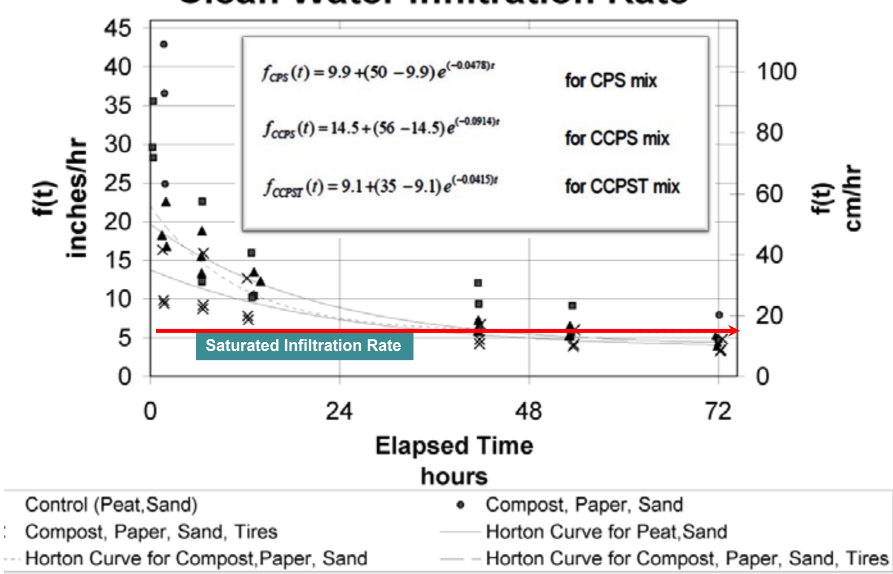
	Sample Depth		
	0-1 cm	4-6 cm	8-10 cm
	mean particle size distribution (D50) in mm		
Peat+ Sand	1.2	0.8	0.25
Compost+Paper+ Sand	0.85	0.75	0.8
Compost+Paper+ Sand+Tires	0.75	0.9	0.85







Clean Water Infiltration Rate

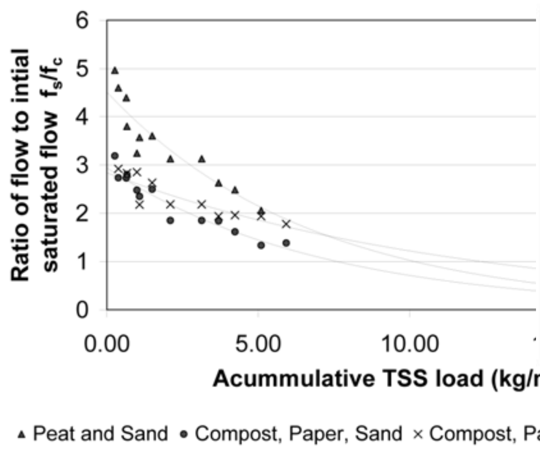


Why Sand-Peat mix needs the longest time to reach saturation?
Why Sand-Comp-Paper-Tire mix reaches its saturation in such a short time?
Why these three different mix media share the same final infiltration rate?

Clogging tests are conducted with the synthetic storm water that can be produced using clean water mixed with street dust and stormwater solids. Based on the annual sediment amount at the LID site, the sediment loadings onto the infiltrometer can be converted into the years of service. The following sediment loads were tested in the laboratory:

Stormwater Application Number	Total		TS mg/l	
	Applied	Cumulative Load	Applied	Cumulative Load
	kg/m2	grams	kg/m2	grams
1	103	12	269	31
2	84	21	242	58
3	71	29	203	81
4	70	37	237	108
5	52	43	224	134
6	260	73	373	176
7	441	123	492	232

Reduced infiltration rate due to clogging



$$\frac{f_s}{f_{C-CPS}} = 4.449\, e^{-0.1361 L_s}$$

$$\frac{f_s}{f_{C-CCPS}} = 2.927\, e^{-0.1369 L_s}$$

$$\frac{f_s}{f_{C-CCPST}} = 2.8303\, e^{-0.0814 L_s}$$

f_s = clogged infiltration rate with storm water
f_c = saturated infiltration rate with clean water
L_s = sediment load applied to LID surface in Kg/m^2

Applications to LID Life-Cycle Clogging Effect

The annual runoff volume, V_o , is generated from the tributary area, A_o, *that has a runoff coefficient, C, and annual precipitation of* P_o.

$$V_o = CP_oA_o$$

The annual sediment load, L_o, depends on mean sediment concentration C_o as:

$$L_o = C_oV_o$$

The annual unit-area sediment load, L_B, to the rain garden's surface area, A_B, is:

$$L_B = \frac{L_o}{A_B} = C_oCP_o\frac{A_o}{A_B}$$

Under a specified accumulated sediment load, L_S, the years of service is calculated as:

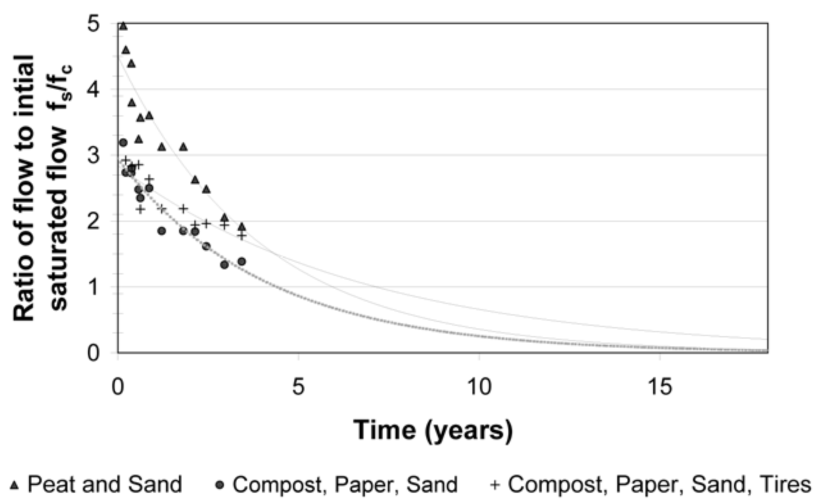
$$N = \frac{L_S}{L_B}$$

For example, a rain garden is designed to have an area ratio of 20 to 1 between the parking lot area and the rain garden area. With C_o = 240 mg/L, A_o/A_B=20, C=0.9, and P_o=0.4 m, the annual unit-area sediment load, L_B, to the rain garden is calculated:

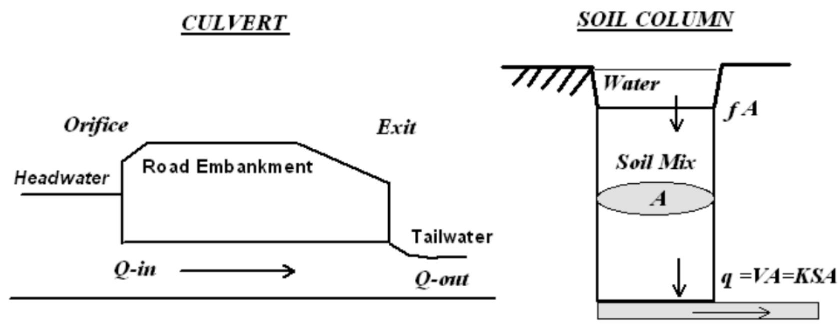
$$L_B = (240mg/l)\times 0.9\times 20\times 0.4 = 1.7kg/m^2$$

The year of service = L_s/L_B

LID Life-Cycle Clogging Study For Denver 's Parking lot

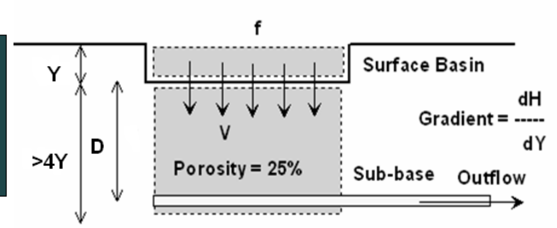


Infiltrating Flow Hydraulics



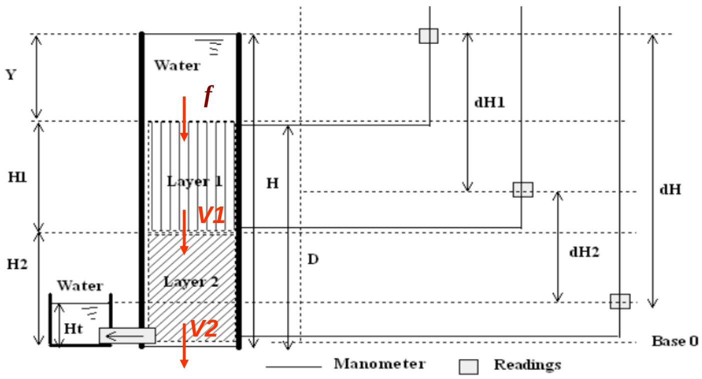
thru-flow = min (inflow, outflow)

OPERATION OF INFILTRATING Detention Basin



- **Unsaturated Operation (Filling process)**
  infiltrating flow stored in the subsurface reservoir (soils)
  one-ft water in the basin = 4-ft soils if the soil porosity =25%
- **Saturated Operation (Depletion process)**
  Infiltrating flow through the saturated column of soils
  infiltrating flow *f* on the surface
  seepage flow V through the soils V = KS
- **Example**: *f* = 4 inch/hr on surface and V = 2 inch/hr through soils
  f > *V* or inflow > outflow
  The entire system is backed up and results in
  (a) standing water on the surface and
  (b) water mounding in the subsurface

Basic Concept for PLD Optimal Design



Basic Principles for the Optimal Design --- Saturated Operation

For the selected drain time T_d and water quality control volume, WQCV

(1) Basin Area	A = WQCV/Y	(Y=12 in)
(2) Sub-layer Thickness	$D = f\,T_d = H_1+H_2$	(T_d=12 hr)
(3) Flow Continuity	$f = V_1 = V_2$	(V= K dH/H)
(4) Energy Consumption	$dH_1 + dH_2 = D+Y$	(Ht =0)

Optimal Design: H_t=0 and $f = V_1 = V_2$

$$f = V_1 = V_2$$

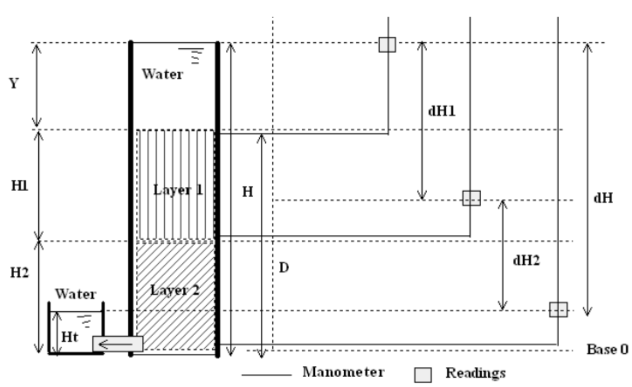
$$V_1 = K_1 S_1 = K_1 \frac{dH_1}{H_1}$$

$$V_2 = K_2 S_2 = K_2 \frac{dH_2}{H_2}$$

$$dH_1 + dH_2 = Y + D$$

$$H_1 + H_2 = D$$

$$H_2 = \frac{(S_1 - 1)D - Y}{(S_1 - S_2)}$$



Normalized Equation

$$\frac{H_1}{D} + \frac{(\frac{f}{K_1} - 1) - \frac{Y}{D}}{(\frac{f}{K_1} - \frac{f}{K_2})} = 1$$

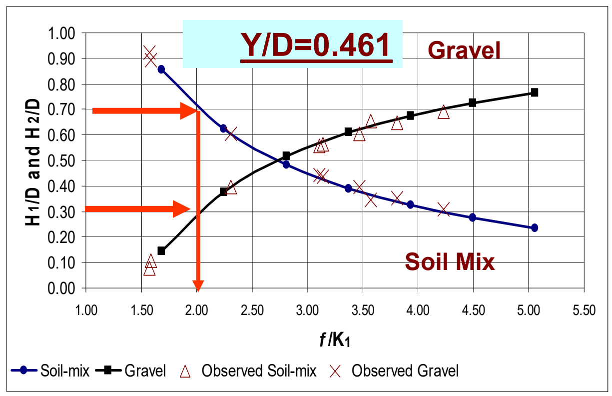
Optimal PLD Sub-Base Design (12-hr Drain)

$$\frac{H_1}{D} + \frac{(\frac{f}{K_1} - 1) - \frac{Y}{D}}{(\frac{f}{K_1} - \frac{f}{K_2})} = 1$$

Given: f = 2.0 in/hr, K_1 = 0.95 in/hr for sand-mix, K_2=25.3 in/hr for gravel, T_d = 12 hours and Y=12 in.

Solution

Thickness
D= T_d f =12*2=24 in
Use D=26 in
f/K_1=2.1
f/K_2=0.079
Use Optimal Eq
H_1/D=0.69
H_1=0.69*26=17.8 in
H_2=0.31*26= 8.2 in



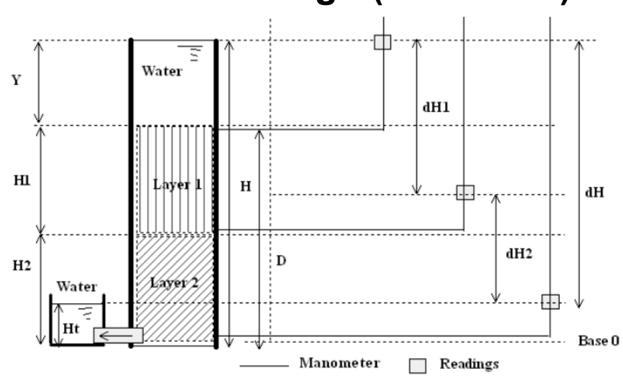
Optimal PLD Sub-base Design (6-hr Drain)

Given:
Y=12 inch
f = 4.5 in/hr
K_1= 2.5 in/hr
K_2= 25 in/hr
T_d = 6 hours

Solution:
D=6*4.5=27 in

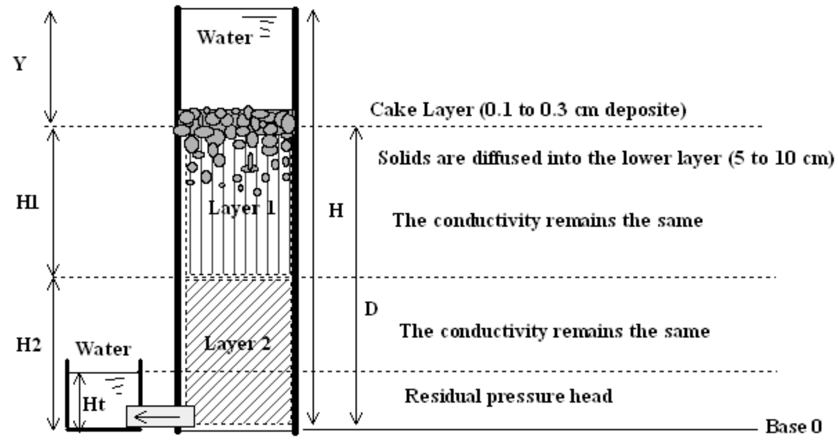
$$\frac{H_1}{D} + \frac{(\frac{f}{K_1} - 1) - \frac{Y}{D}}{(\frac{f}{K_1} - \frac{f}{K_2})} = 1$$

H_1/D=0.791
H_1 = 21.3 in
H_2 =27- 21.3= 5.7 in



f = 4.5 in/hr = V_1 = K_1*dH_1/H_1 = 2.5 *dH_1/21.3
So, dH_1= 38.3 in
f = 4.5 in/hr = V_2 = K_2*dH_2/H_2 =25.0 *dH_2/5.7
So. dH_2 = 0.7 in
Total head available H = 12 + 27 = 39 in
Total consumption = 38.3+0.7= 39 in or Ht =0
Drain time = H_1/V_1 +H_2/V_2 = D/f = 6 hours

Clogging Condition -- Observed in Lab



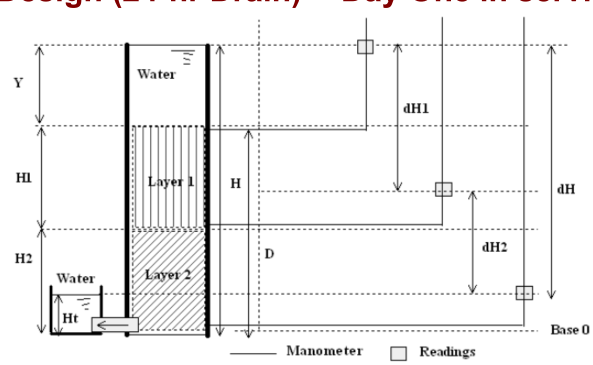
PLD Sub-base Design (24-hr Drain) -- Day One in service

Given:
Y=12 inch
f = 1.5 in/hr >K1
K_1= 0.75 in/hr
K_2= 25 in/hr
T_d = 24 hours

Solution:
D=24*1.5=36 in

$$\frac{H_1}{D} + \frac{(\frac{f}{K_1}-1)-\frac{Y}{D}}{(\frac{f}{K_1}-\frac{f}{K_2})} = 1$$

H_1/D=0.655
H_1 = 23.6 in
H_2 =36- 23.6= 12.4 in



f = 1.5 in/hr = V_1 = K_1*dH_1/H_1 = 0.75 *dH_1/23.6
So, dH_1= 47.2 in
f = 1.5 in/hr = V_2 = K_2*dH_2/H_2 =25.0 *dH_2/12.4
So. dH_2 = 0.8 in
Total head available H = 12 + 36 = 48 in
Total consumption = 47.2 + 0.8= 48 in or Ht=0
Drain time = H_1/V_1 +H_2/V_2 = D/*f* = 24 hours

Clogged PLD Operation (Years in Service: $f > K_1$)

Existing System

Y=12 inch

f = 1.0 in/hr

(reduced, but >K1)

K_1= 0.75 in/hr

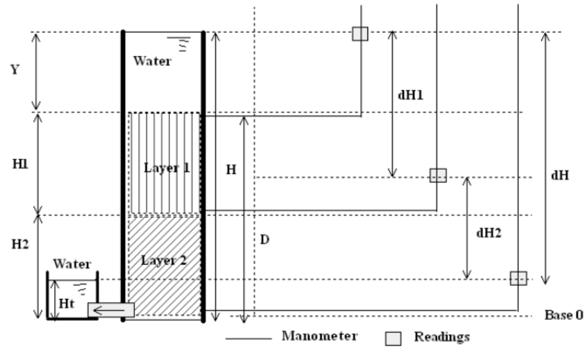
K_2= 25 in/hr

T_d = 24 hours

D=36 in

H_1 = 23.6 in

H_2 = 12.4 in



f = 1.0 in/hr = V_1 = K_1*dH_1/H_1 = 0.75 *dH_1/23.6

So, dH_1= 31.5 in

f = 1.0 in/hr = V_2 = K_2*dH_2/H_2 =25.0 *dH_2/12.4

So. dH_2 = 0.5 in

Total head available H = 12 + 36 = 48 in

Total consumption = 31.5 + 0.5= 32 in

** Residual pressure (mounding) H_t = 48-32= 16 in

** Drain Time = D/f = 36 hours > 24 hours =➔ standing water for 12 hrs

Critical PLD Operation (Many Years in Service: $f = K_1$)

Existing System

Y=12 inch

f = 0.75 in/hr

(reduced to K_1)

K_1= 0.75 in/hr

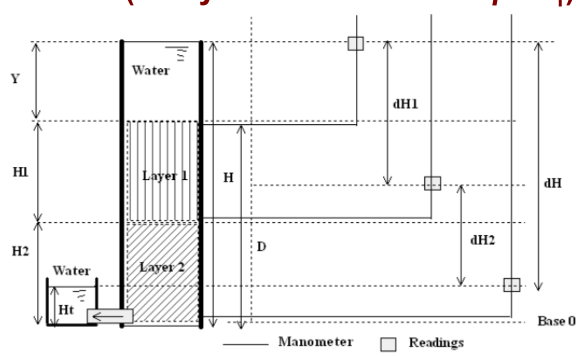
K_2= 25 in/hr

T_d = 24 hours

D=36 in

H_1 = 23.6 in

H_2 = 12.4 in



f = 0.75 in/hr = V_1 = K_1*dH_1/H_1 = 0.75 *dH_1/23.6

So, dH_1= 23.6 in

f = 0.75 in/hr = V_2 = K_2*dH_2/H_2 =25.0 *dH_2/12.4

So. dH_2 = 0.4 in

Total head available H = 12 + 36 = 48 in

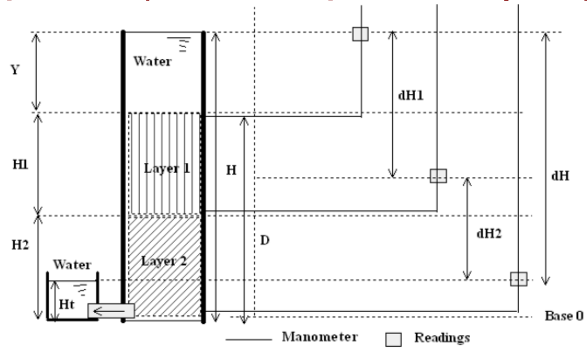
Total consumption = 23.6 + 0.4 = 24 in

** Residual pressure (mounding) H_t = 48- 24= 24 in

** Drain Time = D/f = 48 hours > 24 hours =➔ standing water for 24 hrs

Plugged PLD Operation (Time for Replacement: $f \Rightarrow 0$)

Existing System
Y=12 inch
f = 0.05 in/hr
(reduced to zero)
K_1= 0.75 in/hr
K_2= 25 in/hr
T_d = 24 hours
D=36 in
H_1 = 23.6 in
H_2 = 12.4 in



f = 0.05 in/hr = V_1 = K_1*dH_1/H_1 = 0.75 *dH_1/23.6
So, dH_1= 1.57 in
f = 0.05 in/hr = V_2 = K_2*dH_2/H_2 =25.0 *dH_2/12.4
So. dH_2 = 0.03 in
Total head available H = 12 + 36 = 48 in
Total consumption = 1.57+0.03=1.6 in
** Residual pressure (mounding) H_t = 48- 1.6= 46.4 in
** Drain Time = D/f = 720 hours =➔ standing water on surface

Energy Principle for Seepage Flow – No cap

- $T_d = \frac{Y}{f} = \frac{12\ in.}{1.0\ in/hr} = 12\ hrs$ (10)
- Assume the flow is steady:

$$Q = fA_R = K_s I_s A_R = K_g I_g A_R \quad (11)$$

- total hydraulic head

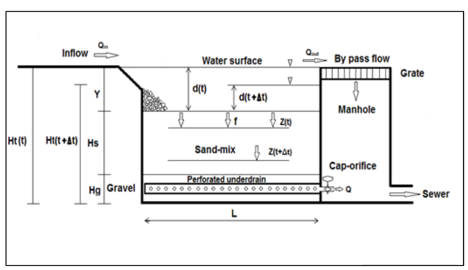
$$H_t = Y + H_s + H_g$$

- Residual head:

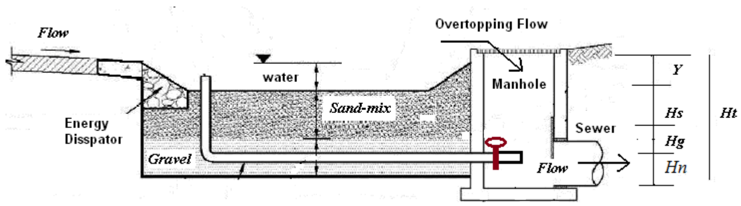
$$\Delta H = H_t - \Delta h_s - \Delta h_g - \Delta h_N$$

$$\Delta h_s = \frac{f}{K_s} H_s$$

$$\Delta h_g = \frac{f}{K_g} H_g$$



Flow adjustment using cap-orifice



To satisfy the principle of energy, the friction loss through the underdrain pipe is computed as:

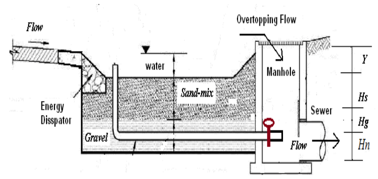
$$\Delta h_N = kL\frac{N^2Q^2}{D^{(16/3)}} \quad (8)$$

in which Δh_N = friction loss in [L] through underdrain pipe, L = pipe length in [L], D= diameter in [L] of underdrain pipe, N = Manning's roughness coefficient, k=4.65 for unit of feet-second or 10.28 for unit of meter-second. The cross section area for the required cap orifice is calculated as:

$$A_o = \frac{Q}{C_d\sqrt{2g(H_t - \Delta h_s - \Delta h_g - \Delta h_N)}} \quad (10)$$

in which A_o = opening area of cap orifice in [L^2], C_d = discharge coefficient, and g= gravity acceleration in [L/T^2]. In practice, the cap orifice must have a diameter smaller than the underdrain pipe.

A rain garden is designed to release the ponding water depth of 12 inch over 4 hours using a flow regulator. The infiltration bed has an flat area of 500 ft^2. The dimensions of filtering system are: *Y=12 inches, H_s=18 inches, H_g=8 inches*. The hydraulic conductivity is 2.5 inch/hr for the sand layer and 25.0 inch/hr for the gravel layer. A cap-orifice is used as the flow regulator. Determine the opening area for the cap-orifice.



$$Q = fA_B = \frac{3.0}{12\times 3600}\times 500 = 0.035 \text{ cfs}$$

$$I_s = \frac{f}{K_s} = \frac{3.0}{2.5} = 1.2 \quad \text{for the sand layer}$$

$$I_g = \frac{f}{K_g} = \frac{3.0}{25.0} = 0.12 \quad \text{for the gravel layer}$$

The energy losses through the sand and gravel layers are calculated as:

$$\Delta h_s = I_s H_s = 1.2\times 18 = 21.6 \text{ inches}$$
$$\Delta h_g = I_g H_g = 0.12\times 8 = 0.96 \text{ inch}$$

Considering the underdrain pipe is described as: D=4 inch, L=25 feet, and N=0.012, the friction loss through the underdrain pipe is:

$$\Delta h_N = 4.62L\frac{N^2Q^2}{D^{(16/3)}} = 4.62\times 25\times\frac{0.012^2\times 0.035^2}{(4/12)^{(16/3)}} = 0.007\ ft = 0.084\ inch$$

With C_d=0.70, the cross sectional area for the cap orifice is calculated as:

$$A_o = \frac{0.035}{0.70\sqrt{2\times 32.2(38-21.6-0.96-0.084)/12}} = 0.0055 \text{ sq ft} \quad \text{or one inch in-diameter.}$$

Bio-Basin with no Cap Orifice

Surface Storage Basin In Bio-Retention)Porous Landscaping Basin)			
A1) Tributary Area to the LID Unit	Area =	10000	sq ft
A2) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	I =	0.60	
A3) Water Quality Capture Volume in depth	WQCV=	0.19	inches
A4) Design Volume: Vol-LID = (WQCV / 12) * Area	VLID	157	cub ft
A5) Design Water Depth	d=	12.00	inches
A6) Surface Area for LID Unit	A-LID	**157.4**	sq ft
B) Sub-Base Geometry for Two-Layered LID Basin			
Thickness of Upper Sand Layer	Hs=	18.00	inches
Hydraulic Conductivity of Sand Layer	Ks=	2.50	inch/hr
Porosity for Upper Sand Layer	Pore-s=	33.00	percent
Thickness of Lower Gravel Layer	Hg=	8.00	inches
Conductivity of Lower Gravel Layer	Kg=	25.00	inch/hr
Porosity for Lower Gravel Layer	Pore-g=	40.00	percent
Available Storage Water Depth=d+Hg*Pore-g	D-design=	21.14	inches >>
C) Enter the Design Infiltration Rate===Start with a guess =>>	f =	**5.00**	**inch/hr**
Seepageg Flow through Porous Pavement Area= f * Ap	Q=	0.0182	cubic ft
Total Energy or Headwater Depth available =Y+Hg+Hs	HT=	38.00	inches
Energy Loss through Upper Layer = f/Ks * Hs	dHs=	36.00	inches
Energy Loss through Lower Layer = f/ Kg * Hg	dHg=	1.60	inches
D) Analysis of Pipe Flow through Perforated Pipe			
Subdrain Pipe Diameter	D=	4.00	inches
Subdrain Pipe Length	L=	100.00	feet
Subdrain Manning's Roughness	N=	0.025	
Subdrain Pipe Flowing Full Velocity = Q/A	V=	0.209	fps
Energy Slope for Flowing Full = (NV)^2/(2.22R^4/3)	Se=	0.000334	ft/ft
Friction loss through the pipe = Se * L*12	dHp	0.401	inches
Energy balance = HT-dHg-dHs-dHp-V^2/64.4 = zero	**Check**	**0.00**	**inches =**
If the energy balnace is not equal to zero, try another infitlration rate.			
E) Drain Time and Dry Time			
Drain time = (d+Hs+Hg)/f	Td=	7.60	hr
Dry time= (Hs+Hg)/f	T-dry=	5.20	hr

Bio-Basin with Cap Orifice

Surface Storage Basin for LID Unit				
A1) Tributary Area to the LID Unit	Area =	10000	sq ft	(input)
A2) Tributary Area's Imperviousness Ratio ($i = I_a / 100$)	I =	0.60		(input)
A3) Water Quality Capture Volume in depth	WQCV=	0.19	inches	
A4) Design Volume: Vol-LID = (WQCV / 12) * Area	VLID	157	cub ft	
A5) Design Water Depth	d=	12.00	inches	(input)
A6) Surface Area for LID Unit	A-LID	157.4	sq ft	
Sub-Base Geometry for Two-Layered LID Basin				
Thickness of Upper Sand Layer	Hs=	18.00	inches	(input)
Hydraulic Conductivity of Sand Layer	Ks=	2.50	inch/hr	(input)
Porosity for Upper Sand Layer	Pore-s=	33.00	percent	(input)
Thickness of Lower Gravel Layer	Hg=	8.00	inches	(input)
Conductivity of Lower Gravel Layer	Kg=	25.00	inch/hr	(input)
Porosity for Lower Gravel Layer	Pore-g=	40.00	percent	(input)
Available Storage Water Depth=d+Hg*Pore-g	D-design=	21.14	inches >>	
Enter the Design Infiltration Rate	f =	*1.00*	*inch/hr*	(input)
Seepageg Flow through Porous Pavement Area= f * Ap	Q=	0.0036	cubic ft	
Total Energy or Headwater Depth available =Y+Hg+Hs	HT=	38.00	inches	
Energy Loss through Upper Layer = f/Ks * Hs	dHs=	7.20	inches	
Energy Loss through Lower Layer = f/ Kg * Hg	dHg=	0.32	inches	
Analysis of Pipe Flow through Perforated Pipe				
Subdrain Pipe Diameter	D=	4.00	inches	(input)
Subdrain Pipe Length	L=	100.00	feet	(input)
Subdrain Manning's Roughness	N=	0.025		(input)
Subdrain Pipe Flowing Full Velocity = Q/A	V=	0.042	fps	
Energy Slope for Flowing Full = (NV)^2/(2.22R^4/3)	Se=	0.000013	ft/ft	
Friction loss through the pipe = Se * L*12	dHp	0.016	inches	
Sizing and Analysis of Cap Orifice				
Headwater Available for Orifice= Ht-dHs-dHg-dHp-V^2/2g	Ho=	30.46	inches	
Orifice Coefficient	Co=	0.65		(input)
Cap Orifice Equivalent Diameter for flow area -- Guessed	**Do=**	**0.31**	inches	**Guess**
Orifice Release	Qo=	0.0044	cfs	
Check if PLD release = orifice flow	**dQ=**	0.0007	**=zero**	**CHECK**
try another cap orifice diameter until dQ =0.				
Drain time	T-drain=	12.00	hrs	
Dry Time	T-dry =	26.00	hours	

Problems with Clogging

Standing Water invites algae growth
Clogged Bottom causes drainage failure
Mosquito Bed introduces public health problems
High Maintenance increases the operational costs

**** Sub-base structure is the key to alleviate these problems.**

Failure Examples of LID Devices

References

- Guo, James C.Y. (2013). "Green Concept in Stormwater Management", Journal of Irrigation and Drainage Systems Engineering, Vol 2, Issue 3, ISBM 2168-9768.
- Guo, James C.Y. and Urbonas B. (2013) " Volume-based Runoff Coefficient", Journal of Irrigation and Drainage Engineering, © ASCE,ISSN 0733-9437/(0). JRNIRENG-S-13-00270.
- Blackler, G. and Guo, James C.Y., (2012). "Field Test of Paved Area Reduction Factors using a Storm Water Management Model and Water Quality Test Site," Vol. 17, No. 8, Journal of Irrigation and Drainage Engineering, © ASCE, ISSN 0733-9437/(0). IRENG-6569, August.
- Blackler, G. and Guo, James C.Y.(2013). "Paved Area Reduction Factors under Temporally Varied Rainfall and Infiltration", ASCE Journal of Irrigation and Drainage Engineering, Vol. 139, No. 2, February 1, 2013
- Guo, James C.Y. (2014). Discussion on "*Storm Centering Approach for Flood Predictions from Large Watersheds*", Journal of Hydrologic Engineering, DOI: 10.1061/(ASCE)HE.1943-5584.0000540, Vol 19, No 1,Jan., pp 272-274
- Guo, James C.Y. (2012). "*Storm Centering Approach for Flood Predictions from Large Watersheds*", Journal of Hydrologic Engineering, Vol. 17, No. 9, September 1,
- Kocman, S. M.,Guo, J. C.Y. and Ramaswami A. (2012)." *Bench-Scale Evaluation of Waste- Incorporated Sustainable Design of Porous Landscape Detention Basins*", ASCE Journal of Environmental Engineering, doi:10.1061/(ASCE)EE.1943-7870.0000391
- Guo, James C. Y. Cheng, Jeff, Wright, L.(2012) "Field Test on Conversion of Natural Watershed into Kinematic Wave Rectangular Planes, ASCE J. of Hydrologic Engineering, Vol. 17, No. 8, August.
- Guo, James C.Y. (2012). "*Off-stream Detention Design for Stormwater Management*", Journal of Irrigation and Drainage Engineering, Vol. 138, No. 4, April 1.
- Guo, James C. Y. Shih, H. M. and MacKenzie K. (2012) "Stormwater Quality Control LID Basin with Micro pool", Journal of Irrigation and Drainage Engineering, Vol. 138, No. 5, May 1.
- Guo, James C.Y. (2012). "*Cap-orifice as a Flow Regulator for Rain Garden Design* ASCE J of Irrigation and Drainage Engineering, Vol. 138, No. 2, February.
- Guo, James, C.Y., Urbonas, B., and K. MacKenzie (2011). "*The Case for a Water Quality Capture Volume for Stormwater BMP*", J. of Stormwater, Oct.
- Guo, James C.Y., Blackler, E G, Earles, A, and MacKenzie, K (2010) "Effective Imperviousness as Incentive Index for *Stormwater LID Designs*" ASCE J. of Irrigation and Drainage Engineering, Vol 136, No12, Dec.
- Guo, James C.Y., Jones, J. and Earles, A.(2010). "Method of Superimposition for Suction Force on Trash Rack", ASCE J. of Irrigation and Drainage Engineering, Vol 136, No11, Nov.
- Guo, James C.Y., and Jones, J. (2010). "Pinning Force during Closure Process at Blocked Pipe Entrance", ASCE J. of Irrigation and Drainage Engineering, Vol 136, No2, Feb.
- Earles, T., Guo, J., MacKenzie, K., Clary, J., and Tillack, S. (2010) "A Non-Dimensional Modeling Approach for Evaluation of Low Impact Development from Water Quality to Flood Control". Low Impact Development 2010: pp. 362-371.
- doi: 10.1061/41099(367)32
- Guo, James C.Y. (2010)"Preservation of Watershed Regime for Low Impact Development using (LID) Detention", ASCE J. of Engineering Hydrology, Vol 15, No 1., January.

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Porous Pavements in UC-Denver Campus