Final Technical Report:

Galveston Beach Rainwater Runoff Investigation

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TERMINOLOGY

Base Flow: The sustained portion of stream discharge that is drawn from natural storage sources, and not affected by human activity or regulation.

Deep Sand: The thickness of the soil over bedrock that is 40 to 60 inches deep.

Catchment: Surface drainage area or a watershed. It also indicates the catching or collecting of water, especially rainfall.

Drainage Area: The total surface area upstream of a point on a stream that drains toward that point. The drainage area may include one or more watersheds.

Flow Accumulation: Calculation of flows into each downslope geographical location or each cell point within a DEM raster.

Green Roof: Extensive vegetated roof covers of typically 6 inches or less in depth that are planted on the roof surface with a purpose of mitigating the rainfall runoff.

Invert: The lowest point on the inside of a sewer or other conduit.

Micropool: A smaller permanent pool that is incorporated into the design of larger stormwater ponds to avoid resuspension of particles and minimize impacts to adjacent natural features.

Pour Points: The point at which water flows out of an area. Usually the lowest point along the boundary of the drainage catchment.

Sandy Loam: Material has 7 to less than 20 percent clay and more than 52 percent sand, and the percentage of silt plus twice the percentage of clay is 30 or more.

Streams: Areas where surface water flows sufficiently to produce a defined channel or bed.

Stream Junctions: Points at which two or greater number of runoff stream tributaries are intersected

Stream Links: Sections of a stream channel connecting two successive junctions, a junction and the outlet, or a junction and the drainage divide.areas where surface water flows sufficiently to produce a defined channel or bed.

Stream Network: A large number of stream links crossing each other to construct the drainage catchment.

Stream Ordering: A process of assigning the numeric order to respective links within a stream network

Time of Concentration: The time for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. It is a measure of how quickly or slowly a watershed will respond to rainfall input.

Trunk Drainage Structures: Large-capacity stormwater pipes or drainage channels.

Watershed: An area of land whose total surface drainage flows to a single point in a stream.

Weep Hole: Opening left in a revetment, bulkhead, or wall to allow groundwater drainage.

OVERVIEW

During rain events, portions of the water falling on land either are collected on surfaces and then filter into the substrate, or produce runoff flow from higher to lower elevations over ground surfaces or through stormwater drains and sewers where such systems are present. Rainwater runoff from barrier islands, such as Galveston Island, can flow toward the ocean or bay side of the island, respectively, depending on local elevation gradients. Once runoff flows reach ocean side beaches, erosion and scouring of the beach can occur, leading to hazardous conditions for residents and visitors, as well as potential problems for coastal structures due to washing out of material and scour formation with the potential to undermine structural foundations (e.g. Galveston Seawall). Furthermore, surface runoff, particularly during the initial stages of a rain event, mobilizes and carries non-point pollutants and bacteria directly to the beach and nearshore waters. Both the erosion and pollution potential of rainwater runoff flows can cause hazardous beach conditions and even lead to beach closures (e.g. due to high Escherichia coli levels) which in turn is detrimental to the coastal tourism industry and residents' health.

This report has been created for the Galveston Park Board of Trustees to establish a knowledge base related to Galveston Island runoff issues and aid further studies into the subject. It includes the quantification of design runoff parameters for three selected island locations as examples for the distinctively different eastern, central, and western island settings. Furthermore, a catalog of potential mitigation strategies is presented in tandem with an assessment of each strategy's applicability and feasibility for Galveston Island.

The beaches along the eastern part of Galveston Island have experienced beach scour channels and unwanted pooling of water due to rainfall inundation and overland runoff discharge to the backshore. Surface imperviousness due to land development and a rapidly flattening surface gradient combined with low infiltration rates of the local beach sand have exacerbated the problem. The runoff stream network assessed for the eastern beach site exhibited a complex runoff flow network and great potential to generate concentrated surface flow channels across the eastern beaches.

The beaches in the central Galveston Island region directly receive runoff discharge from the connected impervious surfaces of the overland paved roads, parking lots, and industrial open lands. The overland flow crossing over the surface of the seawall can reach the backshore with great velocities and little lag time. Consequently, the central beaches have at times experienced deep scour erosion at the base of the seawall structure, channel-cuts through the beach and dune line, and direct exposure to contaminants and solids transported by the surface runoff.

The residential beaches on the west end of Galveston Island have a very flat topography backed by resident neighborhoods. The stormwater runoff from individual plots of land is conveyed primarily via roadside ditches and driveways but there appears to be little or no conveyance system in place to drain overflown water out of the area. The synthetic runoff stream network assessed for the western beach site exhibited high-velocity, concentrated surface flows developed by confluence of runoff tributaries from a wide impervious residence area. Accordingly, frequent ponding/flooding problems were experienced by residents even after moderate rainfall events. Some subdivisions have constructed new sand dunes along the community boundary and concrete outlet pipes were buried underneath as temporary drainage measures. However, due to increased discharge velocities at the drainage outlets, the scour holes created on the beach and dunes continued to enlarge and the drainage pipes have repeatedly experienced structural failures that left broken concrete pieces scattered on the beach, disturbing the public beach use.

Recognizing the rainfall runoff issues confronting Galveston Island, the present study provides initial guidance and an overview of potential stormwater management options that can be used to reduce the flooding potential, mitigate the runoff-related beach erosion and flooding problems, and protect water quality based on the quantification of the peak rainfall runoff discharges. Chapter 1 identifies the runoff "hotspots" among the beaches across Galveston Island for further investigation, and discusses current and potential runoff erosion and pollution problems. Chapter 2 quantifies peak runoff discharge rates for the selected beach sites based on historic rainfall data and runoff catchment hydrology analysis. Chapter 3 provides a technical review of existing structural best management practices implementing low-impact development techniques for rainwater runoff mitigation that is found to be adaptable for the urban coastal settings of Galveston Island. Chapter 4 assesses the feasibility of each runoff mitigation design considered in the present study followed by suggestions for its use on Galveston Island.

1. INTRODUCTION: RUNOFF ISSUES ON GALVESTON ISLAND BEACHES

1.1 Background

In an urban watershed covered with a high percentage of impervious or compacted surfaces, the land area available for infiltration or evapotranspiration of precipitation tends to be reduced, increasing the amount of stormwater available for direct surface runoff. The extensive stormwater drainage systems put in place with urban development collect flows from different lower-number stream channels and overland runoff and re-route them via pipes, gullies, culverts, etc. Consequently, this practice leads to increased drainage density and shortened travel time because the alterations of the stream course in the urban drainage system result in more rapid build-up and higher peaks of runoff discharge at the receiving waterbody (National Research Council, 2008).

The City of Galveston is the United States' largest island community established on a barrier island, bounded by the Gulf of Mexico in the south and Galveston Bay in the north. The regional climate creates periodic tropical storms or hurricanes and other weather events that can be accompanied by intense rainfall. Furthermore, these events can be accompanied by high tides that have the potential to cause severe flooding of the lower elevations on the island (City of Galveston, 2003). The sewer and drainage systems for Galveston Island were primarily designed to drain to the north into Galveston Bay, although a significant volume of stormwater from surface runoff and local outfall drainage systems flows south across coastal beaches to discharge into the Gulf of Mexico. Conventional stormwater conveyance systems that were designed to quickly divert water and reduce the risk of urban flooding can create detrimental environments for flood-prone downstream properties. The flows near outfall pipes discharging onto sandy beaches may scarp the streambank and create scour channels the cut through the beach, causing significant sand loss on the upper beach template and lowering of portions of the beach. These scour channels can pose hazards to beachgoers and increase the risk of injury. Outfall pipes on beaches repeatedly experiencing sand erosion and deposition due to runoff-wave interaction may be clogged with sand or damaged if supporting sand is eroded away. In addition, untreated stormwater tends to carry a relatively high load of pollutants, especially in the initial runoff period. This poses an additional risk to beachgoers and residents and can lead to mandatory beach closures if higher-than-normal levels of pollutant markers are measured in the coastal zone.

The existing drainage system of Galveston Island can be categorized into two separate areas: those drained by storm sewers and those drained via surface drainage. Storm sewers are the primary conveyance system within the area east of Scholes Airport and north of Seawall Boulevard. The stormwater drainage from the western island communities relies on open channel collector systems with culverts and/or bridges and some supplemental sewer systems (City of Galveston, 2003). More distinctively, the overland flow discharging to the oceanside beaches tends to follow one of largely three drainage characteristics, that is, by running down an abrupt seawall structure, as sheet flow crossing the compacted soils and unpaved surface areas, or as a channelized flow conveyed by spill water channels and pipe outlets collecting runoff from low-lying residential areas. Accordingly, issues associated with stormwater runoff and beach outfall can be identified based on the respective drainage site conditions. In the following, three focus areas are identified for further investigation of regional runoff, erosion, and pollution issues.

1.2 Study Sites

1.2.1 Eastern Beaches – Ponding and Scour Channels

The beaches along the eastern part of Galveston Island, such as Stewart Beach and East Beach, have experienced frequent beach closures due to flooding in parking areas and have developed significant beach scour channels even after moderate rainfall events (Figure 1). Some of these beaches front unpaved parking lots and/or sparsely vegetated sand dunes. Beach erodibility is heavily influenced by sediment infiltration capacity. Rises in the level of the beach water table during wet weather tend to demote groundwater filtration and, in turn, promote offshore sediment transport (Grant US, 1948). Compacted soils and unpaved parking areas and driveways also have "impervious" characteristics in that they severely hinder the infiltration of water, even if they are not composed of pavement or roofing material (National Research Council, 2008). Wet sands on a ponded beach created by unwanted pooling of water that receive runoff flows from urban drainage areas become more vulnerable to erosion by both stormwater runoff and wave action.

Figure 1. Photos of rainwater ponding (left) and beach runoff scour channel (right) after a rainfall event at Stewart Beach (Photos by Youn-Kyung Song, date: 02-09-2018).

1.2.2 Central Beaches in Front of Seawall - Scour Erosion and Sand Loss

The central Galveston Island region is comprised of beaches fronted by the Galveston Seawall. The stormwater sewer systems were designed to drain to the north toward Galveston Bay. However, because of insufficient trunk drainage structures (i.e., large-capacity stormwater pipes or drainage channels) and lack of outlet capacities, overflows from the storm sewers run off as surface flow via residential and commercial areas covered with paved roads, parking lots, and industrial open lands. The rainwater over such impervious areas can enter the nearest receiving waterbodies directly because of their quick travel times, producing significant loadings of suspended solids. The nourished beaches in front of Galveston Seawall serve as a receiving basin for such impervious surface runoff flows. When rainwater falls down on bare or sparsely vegetated beach areas over the bluff edge of the seawall, it dislodges soil and other sediments. When the upland overflow is combined with drainage outfall flows through weep holes at various points on the beachside surface of the seawall, it can produce runoff flows with velocities in excess of 1 m/s (3 ft/s) that can excavate deep scour troughs reaching up to 1.2 m (4 ft) in depth (Kelly DeSchaun, email correspondence with Helen S. Young, April 17, 2013). The backshore area of the beach can be severely scoured from runoff flows (Fig. 2). Scour channels can cut through the dune line opening direct pathways for wave action and tide flows. If runoff scour occurs near the foundation of the seawall structure it can expose the toe protection and jeopardize the stability of the seawall.

The overland runoff can also carry man-made or natural contaminants which then directly reach beach goers and aquatic habitats.

Figure 2. Photo of a runoff scour channel at a seawall drainage point on a central Galveston beach (Photo by Galveston Park Board of Trustees, accessed on: 04-17-2013).

1.2.3 Western Beaches – Scour Holes and Drainage Structure Failures

The beaches along Galveston Island's western end (west of the seawall) have a very flat topography (within the 0 – 2 m, or 0 – 7 ft, above NAVD88) and are backed by relatively dense residential neighborhoods in certain areas. The drainage system is designed to carry stormwater north toward and under FM3005 as per City guideline. However, runoff from Gulf-side residential areas (south of Pirates Beach Blvd between 11-Mile and 13-Mile Road) drains via pipes and culverts that terminate at the back of the beach at a normal berm level. Beachfront properties at those locations are situated just beyond the dune line and are under the control of the State of Texas Open Beach policy (General Land Office, 2008). The stormwater runoff from individual neighborhoods is conveyed primarily via roadside ditches and driveways but there appears to be little or no conveyance system in place to drain overflown water out of the area.

The main problem reported by residents is frequent ponding and flooding even after moderate rainfall events, especially in the backdune areas where runoff cannot freely drain toward the beach and Gulf of Mexico. To alleviate this issue, some subdivisions have installed concrete outfall pipes through the dunes at dune walkover locations to directly discharge runoff flow to the beach. However, at the back of the beach near the outfall of such discharge structures, adverse impacts due to local berm and dune scour holes and beach erosion can occur (Figure 3). In addition, the erosion of beach sediment at the mouth of stormwater outfall pipes after rainfall events often leads to structural failure of the pipe inhibiting stormwater discharge altogether. After the most recent hurricane event (Hurricane Harvey, 8/2017) which was accompanied by record-breaking rainfall, new sand dunes were built on the beachside along the community boundary and concrete outlet pipes were buried underneath at several beach locations. However, due to increased discharge velocities at the drainage outlet, the scour holes created on the beach and near the dunes continued to enlarge and pipe structure failures that leave broken concrete pieces scattered on the beach appear to limit public beach use. The deposition of runoff-contaminated sediment at the inlets and outlets of the drainage pipe systems also appears to require regular cleaning and maintenance efforts. In addition, ponded areas at the beach side pipe outfall can become a safety or health issue.

Figure 3. Photo of an example of scour holes and pipe failures at a drainage outfall near Pirates Beach (on the beach side of the dune at "Buccaneer Boulevard"; photo courtesy of Chris Robb, photo received on 01-29-2018).

To further assess the rainwater runoff issues identified across Galveston Island, three specific target areas along the eastern, central, and western beaches were selected for detailed hydrologic analysis. Figure 4 shows locations of each target study area and Table 1 lists the bounding coordinates (Easting and Northing) of each area of investigation in UTM coordinates (Zone 15N).

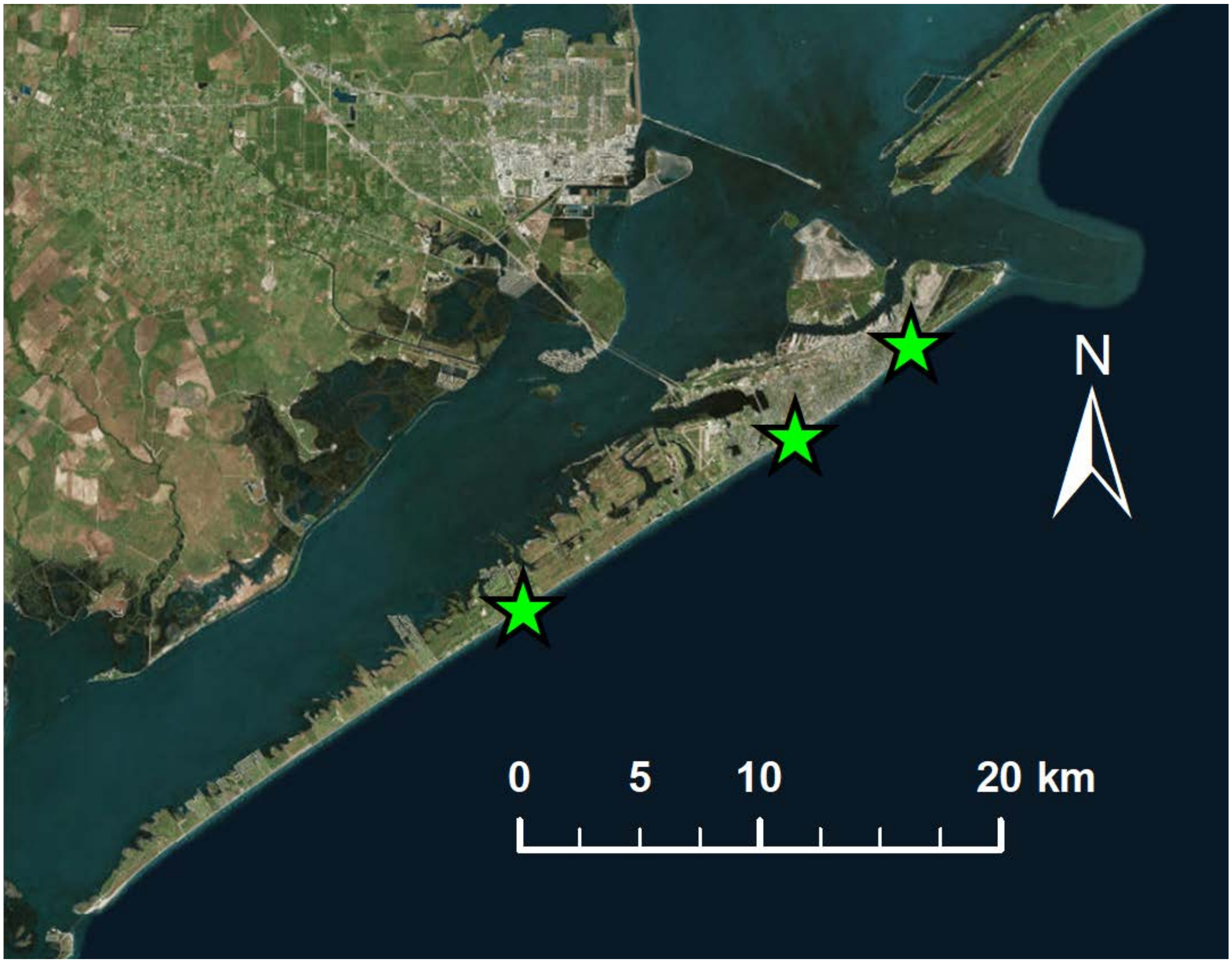

Figure 4. Google Earth™ satellite image of Galveston Island with green stars (☆) indicating three specific rainwater runoff study areas.

Table 1. East and west boundary locations of the selected target study areas.

Beach Location		Easting (m)	Northing (m)
East	East	328690.64	3243379.83
	West	327656.65	3242435.42
Central	East	326039.12	3241245.32
	West	324623.07	3240222.97
West	East	313886.66	3233412.28
	West	310537.02	3231235.67

2. METHODOLOGY: RAINWATER RUNOFF QUANTIFICATION

Hydrologic analysis was performed to estimate the peak rate of the rainfall runoff discharge at the selected locations (i.e., pour points) as a function of the intensity of rainfall, catchment area, and runoff coefficient determined based on land use, cover imperviousness, and hydrologic soil type.

2.1 Rational Method

The Rational Method is being used to analyze the design storm runoff from urban catchments that are not complex and that have a contributing drainage area of 200 acres or less (TxDOT, 2016). It is applicable for drainage with generally uniform surface cover where no significant flood storage is present nearby. The rational method assumes that the rainfall intensity is uniform throughout the duration of the peak-producing rainfall and that the rainfall is distributed uniformly over the contributing drainage area. The rational method does not account for the storage in the drainage area, and thus any available storage is assumed to be filled. The Rational Method is mostly used when only the peak runoff rate evaluation is needed (e.g., storm drain sizing), however, the resulting peak runoff rate can be used to estimate the runoff volume when a local hydrograph is available by assuming that the duration of peak-producing rainfall is also the entire storm duration. The Rational Method, when properly understood and applied, can produce satisfactory results for urban storm drain design and small on-site detention design for sizing of street inlets and storm drains. A more detailed discussion on the underlying assumptions and limitations of the method can be found in various drainage design manuals (Blick et al., 2004; Trommer et al., 1996; UDFCD, 2017). This section describes use of the Rational Method for peak runoff quantification for the urbanized seafront settings across Galveston beaches based on the procedures suggested by the Texas Department of Transportation (TxDOT, 2016).

The rational formula describes the relationship between rainfall intensity and maximum runoff as:

$$Q = \frac{\mathrm{CIA}}{\mathrm{Z}} \qquad (1)$$

where,
Q = maximum rate of runoff (cfs or m^3/sec.)
C = runoff coefficient (non-dimensional)
I = average rainfall intensity (in./hr. or mm/hr.)
A = runoff stream catchment area (ac or ha)
Z = conversion factor, 1 for English, 360 for metric

2.1.1 Runoff Stream Catchment Area, A

Catchment and sub-catchment delineation for the respective study sites was performed based on urban hydrology analysis using ArcGIS (ArcMap 10.4). Essential details of the geospatial analysis processes are presented here.

First, the regional bare-earth elevation information for the respective target site coverage was obtained from the high-resolution (i.e., with a maximum grid size of 1/9 arc-second, approx. 3

meters), Digital Elevation Models (DEMs) provided by the U.S. Geological Survey (http://nationalmap.gov/viewer.html). The spatial elevation differences calculated from the DEM rasters were used to determine the direction, accumulation (quantification of the number of the (synthetic) runoff streams passing each point of the input elevation raster), and synthetic stream network of the surface runoff flow. The resulting stream network essentially displays the connectivity and hierarchy of the surface runoff flows from the overland upstream toward a downstream junction or a pour point (i.e., a common outlet). A junction refers to a point at which two or more runoff stream tributaries intersect and a stream link refers to a mainstream path that connects two successive junctions or a junction and a pour point. A pour point is the common outlet (converging point) of multiple main streams selected at the proper downstream locations considering the site-specific runoff network distribution. Next, stream ordering, the process of assigning ordered numeric values to respective links within a stream network, was performed to identify and classify the types of the streams based on the order numbers assigned to individual runoff tributaries. All links without any tributaries were assigned an order value of 1 and were referred to as first order. The stream order increases when stream links of the same order intersect. Therefore, the intersection of two first-order links will create a second-order link, the intersection of two second-order links will create a third-order link, and so on (Environmental Systems Research Institute, ESRI, 2016). Figure 4 provides an example of the stream ordering and a longest pathway throughout the synthetic stream network within the beachside runoff catchment evaluated for the east project area on Galveston Island.

Overall, the first-order runoff tributaries that were produced imminently by the rainfall water flows and their surface runoff over partially and fully impervious surfaces tended to intersect and become a part of the second- or third-order streams by the time they reached the landward boundary of the beach template. Multiple stream links intersecting on the flat, sandy beach areas tended to form a complex stream network producing concentrated, high-order runoff streams across the backshore surface. In order to assess the accurate stream route, and hence to properly evaluate the longest runoff flow pathways and maximum runoff travel times contingent upon different stream characteristics (i.e., dominant overland flows versus shallow concentrated flows resulting from convergence of multiple links), several mid-pour points were specified as monitoring stations at particular junctions where the second- to third-order links intersected with higher order streams. The local, highest-order runoff streams within a sub-catchment (the area contributing to the generation of the peak runoff rate at the respective mid-pour points) were congregated with the main stream from adjacent sub-catchment areas, and eventually discharged at a common outlet located at the most downstream point of the beach. The common outlets of the main stream runoff were selected as downstream pour points and the boundary assessment for the catchment area was performed to delineate the contributing drainage area to the peak runoff discharge at the selected (mid-) pour points. Finally, the area A, maximum flow length L, and average slope S along the main stream (i.e., the highest-order link) within each of the catchment boundaries were calculated.

Figure 5. Example of stream ordering and a longest pathway throughout the stream network within the beachside runoff catchment for the east project area on Galveston Island. The low-order streams associated with overland flows converge to form higher-order streams on the beachside downstream area (left panel). Note that the transparency of the background DEM raster was adjusted for visual display of the underlying areal image. The longest flow path (dark-red, dotted-line) across the respective stream network (blue lines) is determined (right panel) to later estimate the time of concentration, t_c. Mid-pour points (⊙) are specified at the junctions between the high-order (third- or higher-order) stream links. The peak runoff rates are evaluated at the pour points (▲) located at the downstream end of the converged main streams of the respective runoff stream network.

2.1.2 Time of Concentration, t_c

The evaluation of the t_c was made by following the guideline provided by the Texas Department of Transportation via the Hydraulic Design Manual for urban drainage facilities (Texas Department of Transportation, 2016). This section summarizes the overall procedures for estimating t_c as described in the manual and provides additional information implemented in the present study as needed to assess the site-dependent properties of the hydrologic parameters (e.g., dimensionless retardance coefficient, N) associated with the t_c calculation.

The parameter t_c is the time for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. Therefore, t_c is a measure of how quickly or slowly a watershed will respond to rainfall input. When runoff is computed using the rational method, t_c is the appropriate storm duration that determines the appropriate precipitation intensity. The parameter t_c is a function of length and slope for a particular watercourse.

The time of concentration t_c is commonly estimated by the National Resources Conservation Service (NRCS, 1986) method or by the Kerby-Kirpich method, with the Kerby-Kirpich method being preferred for urban runoff quantification (Roussel et al., 2005). The present study adopted the Kerby-Kirpich approach to calculate t_c as it requires relatively simple, readily available input parameters and the results are readily interpretable.

The Kerby-Kirpich method for estimating t_c is applicable to watersheds with sizes smaller than 150 square miles (9600 acres), the length of overland flow no longer than 1,200 feet (366 meters), main channel lengths between 1 and 50 miles (1.6 and 80.5 km), and main channel slopes between 0.002 and 0.02 (ft/ft) (Roussel et al., 2005).

The Kerby-Kirpich method estimates the total time of concentration by adding the overland flow time t_{ov} and the channel flow time t_{ch}:

$$t_c = t_{ov} + t_{ch} \qquad (2)$$

For small watersheds where the overland flow is an important component of overall travel time, the Kerby method can be used (Roussel et al. 2005).

$$t_{ov} = K(L_{ov} \times N)^{0.467} S^{-0.235} \qquad (3)$$

where,
t_{ov} = overland flow time of concentration, in minutes
K = unit conversion coefficient, with K = 0.828 for English units and K = 1.44 for SI units
L_{ov} = overland-flow length, in feet or meters as dictated by K
N = dimensionless retardance coefficient
S = dimensionless slope of the terrain conveying the overland flow

The upper limit of L_{ov} is 1200 ft (366 meters). A flow path exceeding this upper limit is converted to a concentrated channel. The dimensionless retardance coefficient N for the overland flow varies from 0.02 to 0.80, depending on the land cover characteristics (TxDOT, 2016).

The time component of the concentrated channel flow runoff t_{ch} is estimated as:

$$t_{ch} = KL^{0.770} S^{-0.385} \qquad (4)$$

where,
t_{ch} = time of concentration, in minutes
K = unit conversion coefficient, where K = 0.0078 for English units and K = 0.0195 for SI units
L_{ch} = channel flow length, in feet or meters as dictated by K
S = dimensionless main channel slope

An adjustment to the slope value used for the calculation of t_c is suggested for watersheds with low topographic slope (flat terrain) with average slope less than 0.002 ft/ft (0.2%). This helps avoid unreasonably large values of t_c. The adjusted slope should be S_{low} = S + 0.0005 (Cleveland et al.

2012). In this study, the adjusted slope S_{low} is used for areas with slopes equal to or less than 0.002 ft/ft (0.2%).

2.1.3 Land Use and Impervious Coverage for Drainage Areas

The surface water infiltration potential and perviousness of the composing soils within the respective (sub-) catchment areas were assessed based on available hydrologic soil group (HSG) data (https://websoilsurvey.nrcs.usda.gov/) provided by the U.S. Department of Agriculture-National Resources Conservation Service (USDA-NRCS, 2014). In the USDA-NRCS hydrologic soil group data soils are assigned to one of four groups (A, B, C, or D) or dual soil groups (e.g., A/D) where the soil group 'A' represents the most pervious condition and 'D' the most impervious condition. In the case of the dual soil group, the first letter applies to the drained condition and the second applies to the undrained condition.

The HSG classification shown in Figure 6 demonstrates that the coastal land of Galveston Island was composed largely of either group D or A/D type soils. The undrained condition was assumed, and thus the hydrologic group D soil type was assumed in the peak runoff rate evaluation for the selected coastal Galveston beach sites.

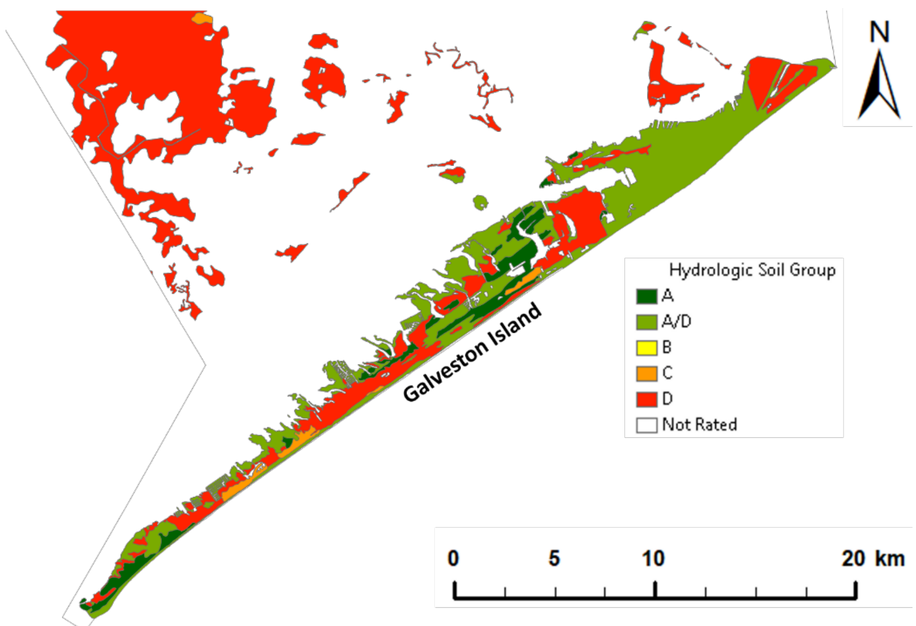

Figure 6. Map of digital soil survey geographic data for Galveston Island developed by the National Cooperative Soil Survey (USDA-NRCS, 2014). Hydrologic soil groups are shown by color as indicated in the legend.

In order to provide the spatial information on the land use and impervious surface distribution, the maximum 30-meter resolution 2011 National Land Cover Data (NLCD) developed by the Multi-Resolution Land Characteristics Consortium (MRLC Consortium, 2011) was used (http://www.mrlc.gov/). The land cover distribution for Galveston Island shown in Figure 7 demonstrates that the land use over the selected study sites can be classified mostly as developed area, barren land, or low- to high-density vegetation area.

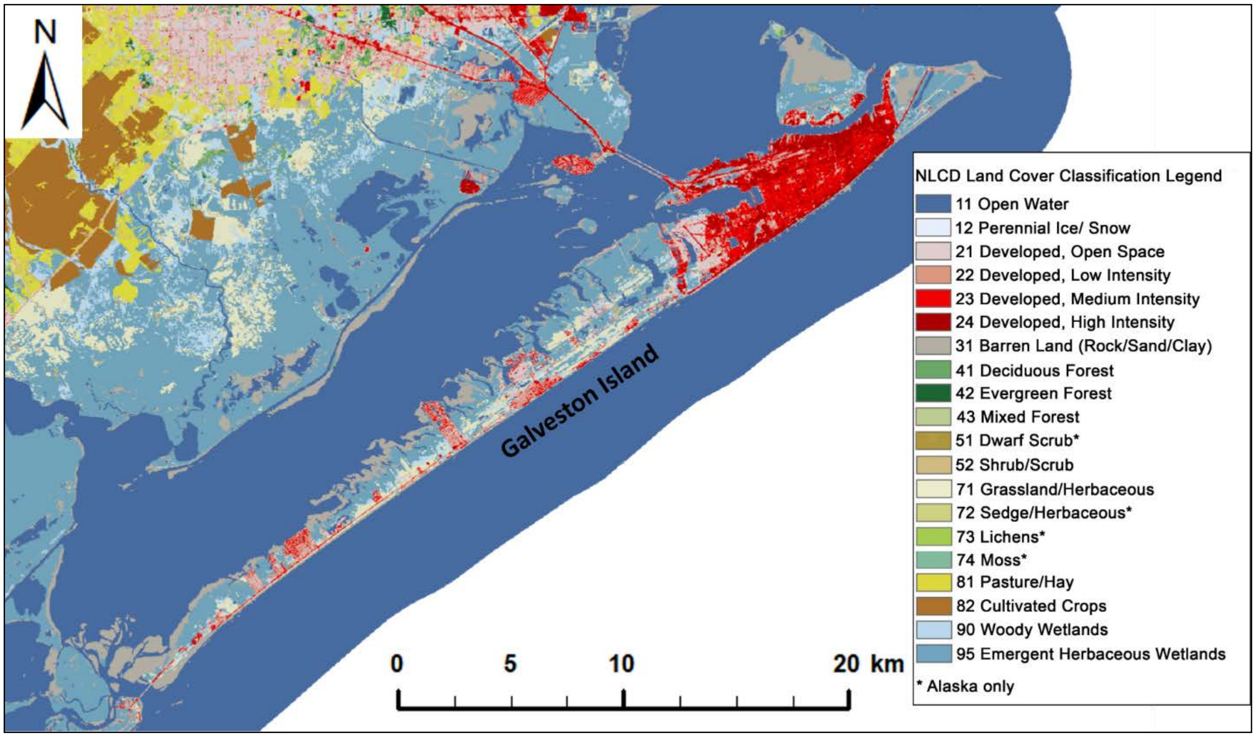

Figure 7. Map showing Galveston Island land cover based on the 2011 National Land Cover Database providing multi-resolution land characteristics (MRLC) for Galveston, TX (MRLC Consortium, 2011).

Information on land use and surface perviousness, soil infiltration capacity, and geographical characteristics of each (sub-) catchment within the areas of investigation were integrated and implemented in the peak runoff calculation using the following procedure.

First, soil composition and imperviousness conditions for hydrologic soil group D was assumed based on the HSG geospatial database (Figure 6). Second, the land use classification provided by the NLCD was simplified by dissolving similar types of land cover into either developed area, barren land, or low- to high-density vegetation area.Third, recommended values for runoff coefficient C and retardance coefficient N for the type “D” hydrologic soils were selected based on the referenced hydrologic design guidelines (TxDOT, 2016; WEF and ASCE, 1992) and were assigned to the respective simplified land cover type. Forth, individual sub-catchment boundaries (delineated based on the DEM database) were used to calculate the fraction of each land use classification within the respective (sub-) catchment areas using the land use composition method developed by Procedures for Delineating and Characterizing Watersheds for Stream and River Monitoring Programs (U.S. EPA, 2018). Figure 8 demonstrates the procedures for (sub-) catchment characterization. Finally, the areal fraction of different land use and perviousness characteristics were used to calculate the areal-weighted values of C and N (see section *Runoff Coefficient, C*) for later use in the peak runoff rate calculation for the respective catchment (pour points) within the target beach site.

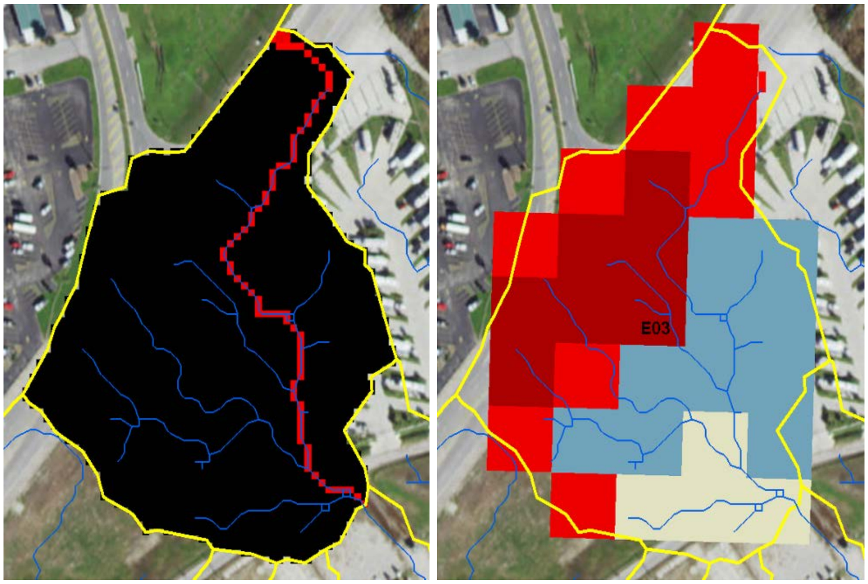

Figure 8. Images demonstrating examples of (sub-) catchment characterization. The area *A*, average slope *S*, and the longest pathway (red dotted lines) are calculated within each (sub-) catchment area delineated from the DEMs (left panel). The areal ratios of the land use (perviousness) characteristics (right panel) are used to calculate the area-weighted runoff coefficient (see Figure 7 for the land use color code index).

2.1.4 Runoff Coefficient C and Retardance Coefficient N

The runoff coefficient C is a measure for the fraction of surface water within a catchment that develops into runoff flow ranging from zero to unity. The values of C reflect the watershed characteristics such as topography, soil type, vegetation, and land use. C approaches unity with an increase in surface slope or surface imperviousness and reduces in value as infiltration capacity increases. The typical values for urban (developed) drainage areas are C = 0.3 – 0.95 for business areas, C = 0.30 – 0.75 for residential areas, C = 0.75 – 0.95 for streets, C = 0.95 – 0.97 for parking lots, and C = 0.05 – 0.20 for deep sand or sandy loam with an average slope less than 2%.

The dimensionless retardance coefficient N is used for the t_c calculation (see section *Time of Concentration*, t_c) is similar in concept to the well-known Manning's roughness coefficient for open-channel flow. However, the values of N for overland flow tend to be considerably larger for a given type of surface. Values of N increase with the surface perviousness from N = 0.02 for pavement to N = 0.8 for dense grass and leafy forest.

The range of values for C and N can be found in various hydrologic design guidelines (TxDOT, 2016; USDA-NRCS, 2010; Li and Chibber, 2008; Roussel et al., 2005; WEF and ASCE, 1992; USDA-SCS, 1947). In the present study, runoff coefficient values of C = 0.70 – 0.90 for the low to high intensity developed areas and open space, C = 0.60 for barren land, and C = 0.20 – 0.25 for vegetated areas are used. The retardance coefficient values based on land use were assigned as N = 0.02 for pavement (roads and streets, parking lots, etc.), N = 0.10 for smooth, bare, packed soil (e.g., backshore), and N = 0.2 – 0.8 for low to high-density vegetation.

For areas with a mixture of land uses, composite runoff and retardance coefficients were calculated by weighting the area of respective land use (TxDOT, 2016):

$$C = \frac{\sum_{j=1}^{n} C_j A_j}{\sum_{j=1}^{n} A_j} \text{ and } N = \frac{\sum_{j=1}^{n} C_j A_j}{\sum_{j=1}^{n} A_j} \tag{5}$$

where,
C = weighted runoff coefficient
C_j = runoff coefficient for sub-catchment area j
N = retardance runoff coefficient
N_j = retardance coefficient for sub-catchment area j
A_j = sub-catchment area for land cover j (ft^2)
n = number of distinct land uses

Tables 2, 4, and 6 show the values of C calculated based on the collected geospatial information.

2.1.5 Annual Exceedance Probability (AEP) or Frequency of Rainfall Events

An annual exceedance probability (AEP) represents the likelihood of a specified intensity of a rainfall event to reoccur within a given number of years. The frequency of a specific rainfall event is commonly described in terms of the recurrence time interval, or annual recurrence period (ARI), and related to the AEP as AEP = 1/(recurrence interval in years). For example, a rainfall event that has a 1% chance of being equaled or exceeded in any year at a given location is called a 1% exceedance probability event. In the present rainfall peak runoff estimation, the two (2), five (5), ten (10), twenty-five (25), and one-hundred (100) year rainfall events were evaluated as suggested by several hydrology and hydraulic manuals published for Texas cities (e.g., City of Round Rock, 2018; Harris County Flood Control District, 2009).

2.1.6 Average Rainfall Intensity, I

The rainfall intensity (I) is the average rainfall rate in in/hr for a specific rainfall duration and a selected return frequency. When used in the Rational Method, the duration is assumed to be equal to the time of concentration. On the other hand, the minimum duration to be used for computation of rainfall intensity for the Rational Method is 5 minutes for urbanized areas and 10 minutes for areas that are not considered urban (TxDOT, 2016; UDFCD, 2017). Therefore, in the present runoff evaluation, if the time of concentration computed for the catchment area is less than 5 minutes, it was adjusted to t_c = 5 minutes for the rainfall intensity calculation.

For drainage areas in Texas, the rainfall intensity can be determined using the power-law model, which is known as a rainfall intensity-duration-frequency (IDF) relationship:

$$I = \frac{b}{(t_C + d)^e} \tag{6}$$

where,
I = design rainfall intensity (in./hr.)
t_c = time of concentration (min)
e, b, d = IDF coefficients for specific return frequencies

The coefficients used to determine the rainfall intensity are based on rainfall frequency-duration data contained in the Atlas of Depth-Duration-Frequency of Precipitation of Annual Maxima for Texas (Asquith and Roussel, 2004; Cleveland et al., 2015; TxDOT, 2016). The rainfall intensities used as input for the peak runoff evaluation in the present study were calculated using the most recently updated IDF coefficients evaluated for Galveston County, TX (Cleveland et al., 2015). Table 2 presents the rainfall IDF coefficients in the power-law model evaluating the rainfall intensity for Galveston County, TX as used in the present study.

Table 2. Rainfall IDF coefficients for Galveston County, TX

AEP (in percent)	ARI (in year)	b	d	e
50 %	**2**	58.3	11.04	0.7839
20 %	**5**	70.47	12.6	0.7636
10 %	**10**	77.97	13.38	0.7496
4 %	**25**	91.45	14.79	0.743
2 %	**50**	99.26	14.85	0.7308
1 %	**100**	115.89	16.5	0.7295

Once the design storm recurrence frequency or AEP is selected, the rainfall intensity I can be determined for a given rainfall duration or t_c for the application of the Rational Method. Table 3 provides the general values of I calculated for the assumed duration varying from 5 minutes to 2880 minutes (48 hours). In general, rainfall intensity increases with increased return interval and decreases with rain event duration. When used in the Rational Method, the duration is assumed to be equal to the time of concentration (t_c). t_c was assessed based on the characteristics of each catchment area (*i.e.*, dependent on L, S, and N).

Table 3. Rainfall Intensities calculated with the power-law model for Galveston County, TX

Duration	Rainfall Intensity (in/hr)					
(min)	2-YEAR	5-YEAR	10-YEAR	25-YEAR	50-YEAR	100-YEAR
5	5.35	6.52	7.34	8.42	9.49	10.61
10	5.35	6.52	7.34	8.42	9.49	10.61
30	3.17	4.02	4.62	5.42	6.16	7.04
60	2.06	2.67	3.12	3.71	4.24	4.90
120	1.28	1.69	1.99	2.39	2.76	3.21
180	0.95	1.27	1.51	1.82	2.11	2.46
360	0.56	0.77	0.92	1.12	1.31	1.53
720	0.33	0.46	0.55	0.68	0.80	0.94
1440	0.19	0.27	0.33	0.41	0.48	0.57
2880	0.11	0.16	0.20	0.25	0.29	0.35

3. RESULTS: PEAK RUNOFF RATE, Q

The peak runoff rates were evaluated in the units of water volume per unit second (ft^3/s) at selected pour points. The pour points are specified at the most downstream outlets of the mainstream highest-order links connected from the first-order overland runoff tributaries within each target beach site.

3.1 East Galveston Site: Stewart Beach

Peak runoff rates were estimated at select locations on Stewart Beach as the east Galveston project site. Beaches in this region are affected by surface runoff from both Seawall Boulevard veering to the north and its seaside overland area that is composed of impervious surfaces such as parking lots, buildings and driveways, sparsely vegetated areas, and barren land. Figure 9 shows the synthetic stream network and discharge catchment delineation for rainfall runoff flows on Stewart Beach. It demonstrates the complexity of the runoff stream network as the overland flow initiated from Seawall Boulevard or parking lots over steep-gradient, impervious surfaces converge on the open area between beach and seawall, producing concentrated flow channels. Therefore, sub-catchment delineation (thin, gray lines in Figure 9) was performed for detailed routing in order to evaluate the longest runoff flow pathway L and hence, the time of runoff flood concentration, t_c, within each catchment area. The peak runoff rates against the assumed rainfall events were calculated at the downstream pour points of each individual catchment area for each AEP. Locations of the assumed pour points (E05, E06, E25, E29, E33, and E36) and runoff parameters (A, C, S, L, t_c) characterizing each runoff catchment area are provided in Table 4. Locations of each pour point are given in Universal Transverse Mercator (UTM) coordinates in meters (Zone 15N).

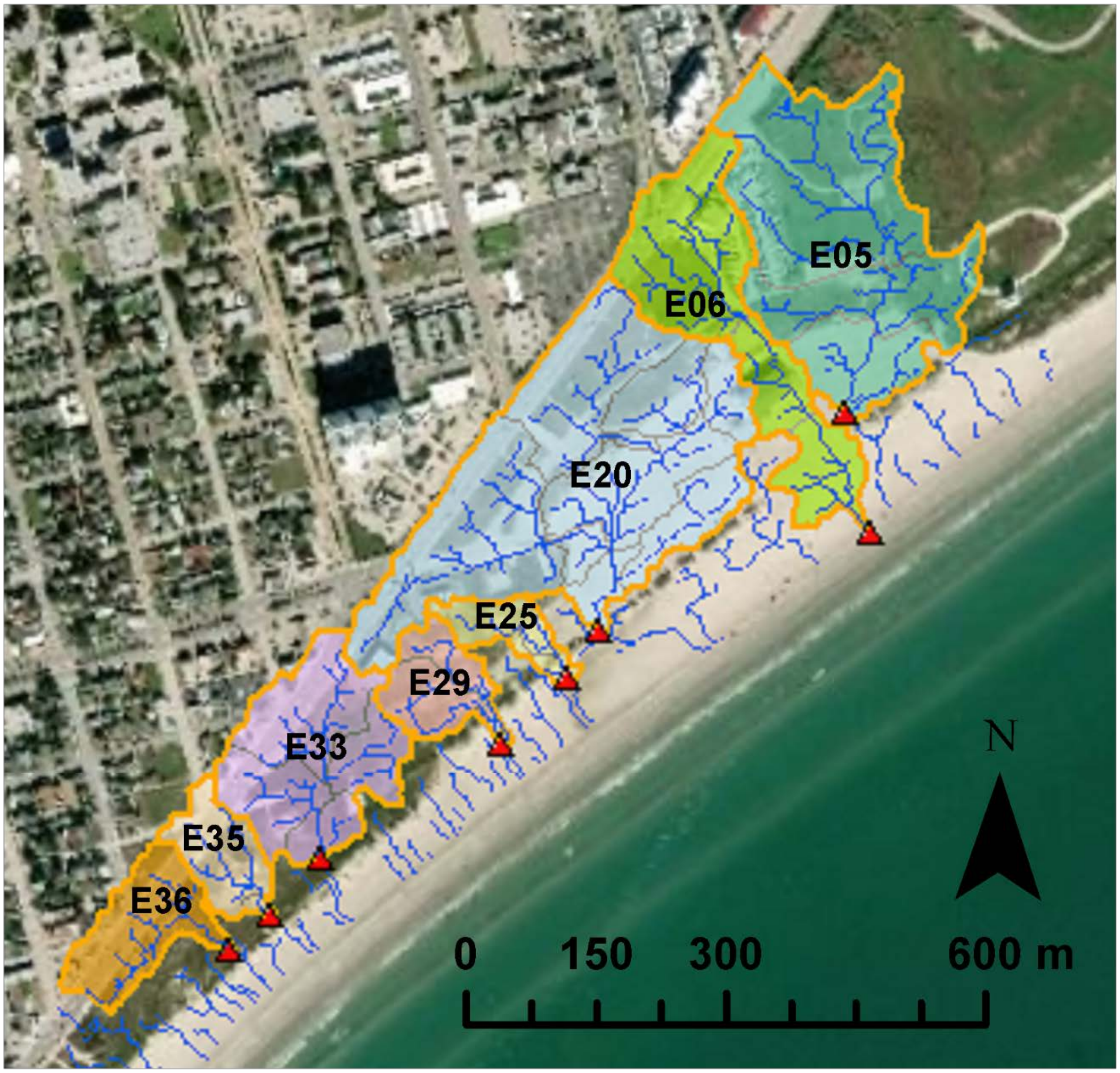

Figure 9. Stream network and discharge catchment delineation for rainfall runoff flows on Stewart Beach overlaid on aerial image. The catchment boundaries (orange lines) delineate the contributing area to the peak runoff discharge at respective downstream pour points (▲) where the most high-order stream links within each synthetic stream network (blue lines) converge. Sub-catchment delineation (grey lines) was performed as necessary for flow routing across the complex runoff stream network associated with overland and channelized flows initiated from different locations within respective catchment areas.

Table 4. Locations of chosen pour points and runoff parameters for the Stewart Beach catchment.

ID	**Easting** (UTM, m)	**Northing** (UTM, m)	**A** (m2)	(ac.)	**C**	***S*** (%)	***L*** (m)	(ft)	t_c t_{ov}	t_{ch} (min)	$t_{ov} + t_{ch}$
E05	328420.19	3243300.65	74954	18.5	0.389	1.33	645	2115	28.7	13.2	41.8
E06	328374.41	3243297.27	43023	10.6	0.548	1.47	618	2028	32.4	11.4	43.8
E20	328133.92	3243046.51	102799	25.4	0.724	1.49	676	2218	30.4	9	39.4
E25	328097.91	3242994.93	7832	1.9	0.874	1.91	248	812	11.1	-	11.1
E29	328019.83	3242916.85	12944	3.2	0.856	2.43	294	965	12.3	-	12.3
E33	327812.16	3242783.34	37773	9.3	0.9	2.39	335	1100	12.9	-	12.9
E35	327755.94	3242718.12	11748	2.9	0.9	2.81	224	734	9.6	-	9.6
E36	327707.52	3242679.17	17315	4.3	0.9	2.8	284	933	7.5	-	7.5

The regional DEM revealed that the surface gradients are significantly higher in areas near Seawall Boulevard compared to lower values near the dunes. The average slope changes from S = 2 – 3% within the sub-catchment adjacent to Seawall Boulevard to S = 0.25 % or lower at the downstream pour points near the dunes. This implicates that the overland runoff flow associated with impervious land cover and steep surface gradients will be dissipated rapidly by creating local scour holes and scour channels as it runs into the flat backdune terrain. In addition, the creation of inundation pools in backdune areas during and soon after rainfall events is expected where damping of floodwater flows over the flat, fine sand beach (i.e., low conveyance and infiltration capacities) is combined with the effect of the rapid overland runoff inflow.

The calculated peak runoff rates are presented in Table 5. The greatest peak runoff rates are estimated at the discharge point E20. High-order stream links can lead to concentrated runoff channels. Therefore, the high-order stream path location is indicative of sites vulnerable to runoff channel formation and flooding. On the other hand, notably high runoff rates were predicted on the west-side catchments (e.g., E33 – E35) in spite of their significantly smaller contributing areas (e.g., compare E33 to E20). The reason is that in this narrow beach region the surface runoff from Seawall Boulevard converged further upland before traveling across the steep, sparsely vegetated area. The large fraction of impervious surface area combined with the steep gradient across the runoff pathway and the short travel distance toward the down pour (i.e., small t_c) contributed to this high rate of runoff discharge at the beach end point. Therefore, local scour hole creation and rapid increase in the level of contamination in the water and beach soil near the discharge points may be the primary runoff issue in these narrow-beach areas.

Table 5. Peak runoff rates calculated at varying rainfall intensities for the Stewart Beach site.

	Q = CIA (cfs)					
	2-year	5-year	10-year	25-year	50-year	100-year
E05	18.7	24.0	27.7	32.8	37.4	42.9
E06	14.7	18.9	21.9	25.9	29.5	34.0
E20	49.6	63.5	73.4	86.6	98.7	113.3
E25	8.7	10.6	12.0	13.8	15.5	17.4
E29	13.5	16.6	18.8	21.6	24.4	27.4
E33	40.6	49.9	56.5	65.1	73.5	82.6
E35	14.2	17.3	19.4	22.3	25.1	28.0
E36	22.7	27.4	30.7	35.1	39.4	43.9

3.2 Central Galveston Site: Beach fronted by 53rd Street

Peak runoff rates for the central Galveston project site were estimated at a select location in the south of Seawall Boulevard between two groins in the vicinity of 53rd Street. Figure 10 shows the synthetic stream network and catchment delineation for surface runoff flows across the site. Locations of the assumed pour points and runoff parameters characterizing each runoff catchment are provided in Table 6.

The runoff streams in this area discharge to the beach perpendicular to the local orientation of Seawall Boulevard. Most of the runoff streams were found to originate from the seawall roadways or seaward end of the sidewalk. However, where Seawall Boulevard is connected to the parking lots of beach resorts or business suites with large impervious footprints, the runoff initiates from further inland (e.g., C03 – C06 and C09). Any beach site connected directly with Seawall Boulevard via the seawall is affected by direct runoff from the overland flow without necessarily converging to develop concentrated channel flows. These direct overland flows usually only comprise of low-order stream runoff (i.e., 1st or 2nd order streams) on the beaches fronting the seawall with relatively small contributing areas (less than approx. 1 acre). However, these beach sites are affected by the direct runoff from Seawall Boulevard with very little travel distance and associated time lag. In some cases, the time of concentration t_c is less than 5 minutes. Since the minimum t_c allowed for the application of the Rational Method (TxDOT, 2016; UDFCD, 2017) is 5 minutes, any t_c values below that threshold were set to t_c = 5 minutes (e.g., C1 and C3). The cross-shore surface gradients of these beach areas are significantly higher (S > 3%) compared to the east Galveston project site. Elevation change between the base of the seawall and the mean water line occurs from ~7 to 0 feet NAVD88 within only a few tens of meters (<< 100 m or 328 ft) of dry beach extent. This means that the overland runoff streams over Seawall Boulevard can rapidly produce the peak rainfall discharge at the seawall outfall which, in turn, can lead to high-velocity flow over the steep-gradient beach area causing deep scour features.

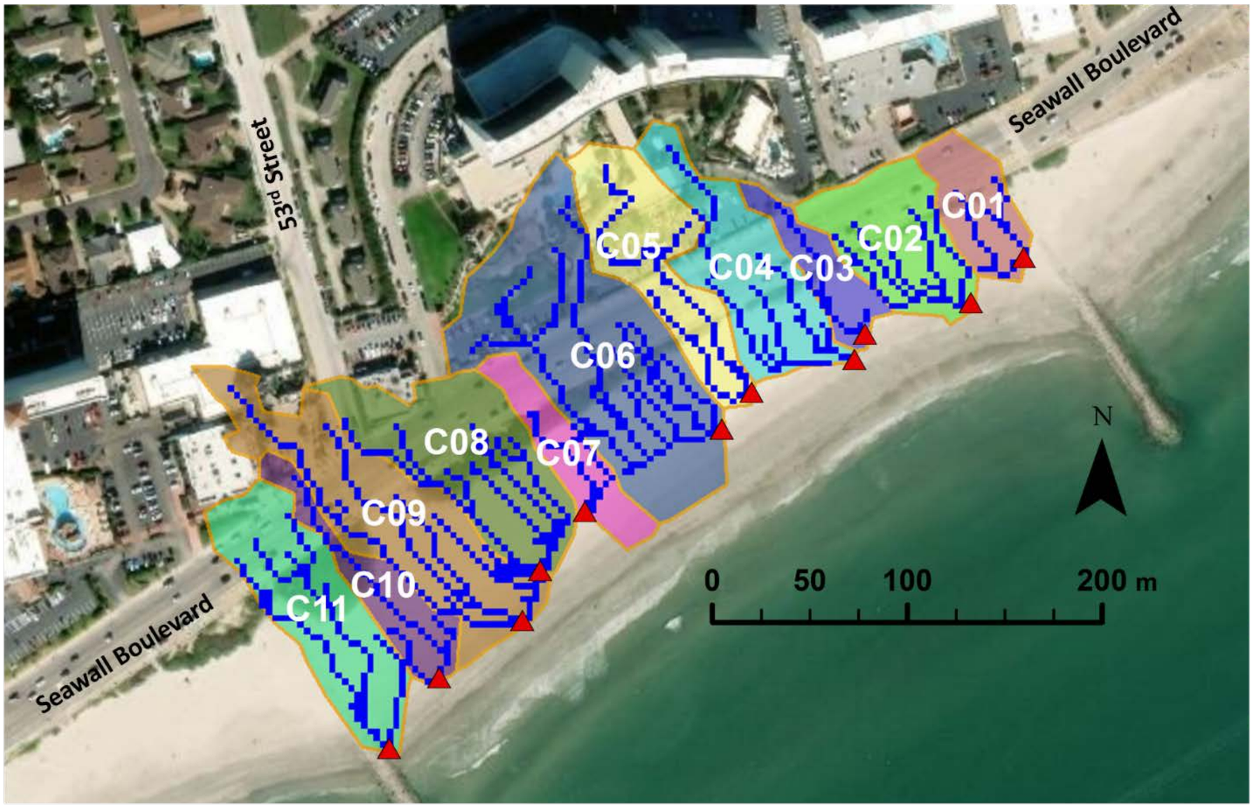

Figure 10. Aerial image and overlay stream network and discharge catchment delineation for rainfall runoff flows at the central Galveston project site (in the vicinity of 53rd Street). The catchment boundaries (orange lines) delineate the contributing areas to the peak runoff discharge at respective downstream pour points (▲) where the synthetic runoff streams (blue lines) running in parallel over the beach connect.

Table 6. Locations of the assumed pour points and runoff parameters characterizing the stream catchment at the central Galveston project site.

ID	Easting (UTM, m)	Northing (UTM, m)	A (m^2)	A (ac.)	C	S (%)	L (m)	L (ft)	t_c: t_{ov} (min)	t_c: t_{ch} (min)	t_c: $t_{ov} + t_{ch}$ (min)
C01	323574.16	3239462.00	2592	0.6	0.83	3.99	98	320	4.2	.	**5.0**
C02	323547.13	3239439.00	4433	1.1	0.83	3.46	124	407	6.7	.	6.7
C03	323492.72	3239423.00	2012	0.5	0.90	4.22	124	407	4.6	.	**5.0**
C04	323487.41	3239409.75	5414	1.3	0.86	4.81	193	634	6.8	.	6.8
C05	323434.69	3239393.00	5213	1.3	0.90	6.24	218	714	5.5	.	5.5
C06	323419.44	3239374.00	12054	3.0	0.77	4.91	235	772	7.9	.	7.9
C07	323349.19	3239331.75	2536	0.6	0.90	2.56	164	537	5.9	.	5.9
C08	323326.25	3239301.75	6717	1.7	0.90	3.09	201	660	6.3	.	6.3
C09	323317.16	3239276.00	7934	2.0	0.78	2.35	235	772	10.7	.	10.7
C10	323274.50	3239246.75	2796	0.7	0.75	2.02	235	772	12.4	.	12.4
C11	323248.88	3239210.25	6099	1.5	0.71	3.05	193	634	11.0	.	11.0

The peak runoff rates evaluated for the central Galveston project site are presented in Table 7. As the regional runoff streams tend to run in parallel down toward the Gulf (Figure 10), the peak runoff rates were evaluated at the assumed downstream conjunctions where the runoff streams converge due to the ground gradient near the water line (this analysis disregards any potential seawater interference). The calculated peak runoff rates are consistently low for the individual catchment zones. However, in this region the runoff parameters of primary concern are the short length of the maximum travel path, L, and the short duration of the runoff concentration, t_c, (due to small L and large S). The regional runoff issue can be classified as significant local scour hole creation and increased soil erosion due to strong drop velocities of the runoff flows from the top of the seawall down to the beach base. This process can undermine the seawall structure over time. On the beach, the high-velocity runoff streams can create scour channels providing a direct flow pathway to the nearshore environment for runoff pollutants, with the potential to threaten the safety and health of beach users and local ecosystem.

Table 7. Peak runoff rates calculated at varying rainfall intensities for the central Galveston project site.

	Q = CIA (cfs)					
	2-year	5-year	10-year	25-year	50-year	100-year
C01	3.5	4.2	4.7	5.3	6.0	6.6
C02	5.5	6.6	7.4	8.5	9.5	10.6
C03	3.0	3.5	3.9	4.5	5.0	5.5
C04	7.0	8.4	9.4	10.7	12.0	13.4
C05	7.5	9.0	10.0	11.3	12.7	14.1
C06	13.4	16.1	18.1	20.7	23.3	25.9
C07	3.6	4.3	4.8	5.4	6.1	6.8
C08	9.3	11.2	12.5	14.2	16.0	17.7
C09	8.0	9.7	11.0	12.6	14.2	15.9
C10	2.5	3.1	3.5	4.1	4.6	5.2
C11	5.5	6.8	7.6	8.8	9.9	11.1

3.3 West Galveston Site: Pirates Beach

Pirates Beach was chosen as the west Galveston Island project site. Peak runoff rates were estimated at select beach locations for the Pirate Beach neighborhood located south of FM 3005, between Pirates Drive and Rageur Boulevard, subtended by 12 Mile Road. Figure 11 shows the synthetic stream network and catchment delineation for surface runoff flows across the Pirates Beach beachfront residential area. Locations of assumed pour points and runoff parameters characterizing each runoff catchment are provided in Table 8. Note that t_c for catchment W1 has been adjusted to t_c = 5 minutes to conform with the Rational Method as explained earlier.

The synthetic stream network indicates that the surface runoff from the northern and southern residential areas in this region flows toward the road Grand Terre (red dotted line in Figure 11) to produce a high-order, concentrated stream link before being conveyed toward the beach for discharge. Based on the surface elevation information form the DEMs, the initial surface runoff from individual households is conveyed through high-gradient (S ≥ 3%) impervious driveways, residential parking lots, and open roadside ditches. The ground gradients along Grand Terre are milder (S < 1%). The high-order, concentrated runoff streams are conveyed toward the beach mostly via roadside ditches and swales across the beachfront residential areas, implicating high-velocity channelized flows. These concentrated runoff flows then drain onto the beach through beach access openings or outlet pipes buried underneath walkover dunes.

The peak runoff rates evaluated for the Pirates Beach site are presented in Table 9. The most significant peak runoff rates were estimated at the pour points of catchment W02 and W04. The concentrated runoff channels formed by the highest 4th order stream links spanned more than 38% (853 ft) and 67% (1427 ft) of the respective longest runoff travel distance (L). Potential runoff problems at this site include rainfall water flooding across Grand Terre where the runoff from surrounding residential areas converges and accumulates on the low-gradient ground. The runoff converging on Grand Terre is then conveyed toward the beach along the limited discharge route provided by roadside ditches and swales that were designed with a significant gradient (S ≥ 3%). The resulting high-velocity, concentrated flow can cause overflow and channel scour across the beachfront neighborhood streets. High concentration of suspended solids and debris can cause blockage of outlet ditches and culverts and can scour the substrate supporting pipe inlets and outlets, resulting in pipe failure. This concentrated runoff discharge can also contaminate the water and soils on the beach side discharge points.

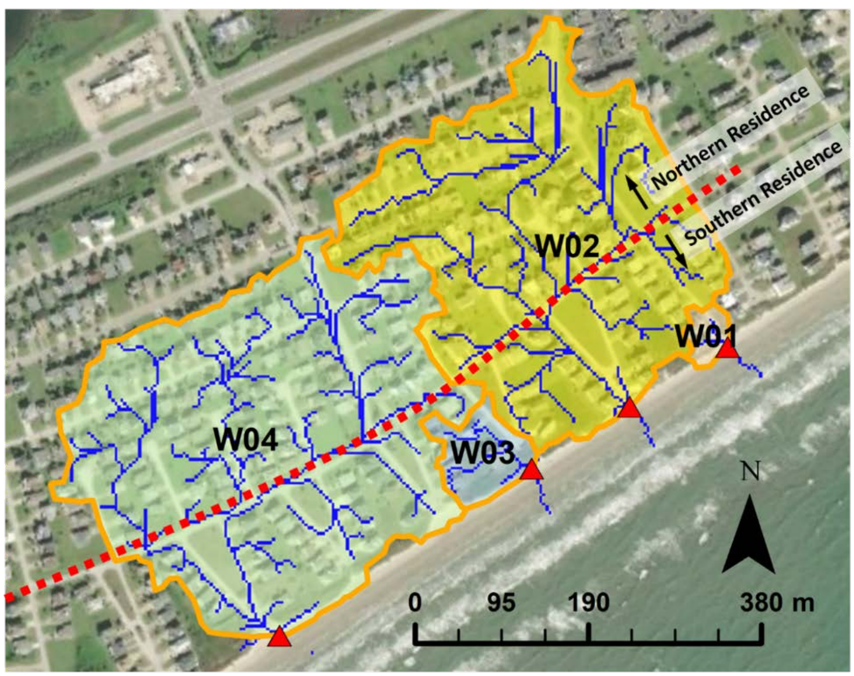

Figure 11. Stream network and discharge catchment delineation for rainfall runoff flows over portions of the Pirates Beach residential area. The catchment boundaries (orange lines) delineate the areas contributing to the peak runoff discharge at the respective downstream pour points (▲) where the most high-order stream links within each synthetic stream network (blue lines) converge. Sub-catchment delineation (grey lines) was performed as necessary for flow routing across the complex runoff stream network associated with overland and channelized flows initiated from different locations within respective catchment areas. The dotted-red line marks the road Grand Terre.

Table 8. Locations of the assumed pour points and runoff parameters characterizing the stream catchment at the Pirates Beach site.

ID	Easting (UTM, m)	Northing (UTM, m)	A (m^2)	A (ac.)	C	*S* (%)	*L* (m)	*L* (ft)	t_c: t_{ov} (min)	t_c: t_{ch} (min)	t_c: $t_{ov} + t_{ch}$ (min)
W01	312276.13	3232337.00	2611	0.6	0.83	1.95	73	241	**4.4**	.	**5.0**
W02	312061.13	3232203.50	110908	27.4	0.76	1.31	681	2235	10.8	8.7	19.5
W03	311784.31	3232020.75	8537	2.1	0.74	1.54	177	581	6.9	.	6.9
W04	311784.31	3232020.75	130734	32.3	0.79	1.49	649	2130	9.8	7.6	17.4

Table 9. Peak runoff rates calculated at varying rainfall intensities for the Pirates Beach site.

	Q = CIA (cfs) 2-year	5-year	10-year	25-year	50-year	100-year
W01	3.6	4.2	4.7	5.4	6.0	6.6
W02	83.4	104.0	118.6	138.0	156.2	177.0
W03	9.5	11.4	12.8	14.6	16.4	18.2
W04	108.4	134.6	153.3	177.9	201.1	227.5

In the appendix, the peak runoff rates presented in Table 5, Table 7, and Table 9 are represented by the measure of the surface inundation depth per unit hour (in./hr), consistent unit for the rainfall intensity, for the purpose of aiding translating the expected runoff quantity in comparison to the input rainfall intensity. The runoff inundation rates (in./hr) were rigorously approximated by dividing the peak runoff rates (ft^3/s) evaluated at each pour point by the area of respective drainages (A). It assumed that the surface runoff that could produce the peak runoff rate at a pour point selected in the downstream was uniformly distributed over the upstream contributing drainage area, which is not necessarily true. Therefore, it urges caution against dictating the runoff flooding depth directly from the results presented in Table A- 1, Table A- 3, and Table A- 5.

Additionally, the peak runoff rates (Q) was also converted to the potential hydraulic power (P_h) that can be produced by the surface water runoff discharging at each downstream pour point, according to the following relation.

$$P_h = \rho g \eta Q H / 1000 = \rho g Q H_{100\%} / 1000 \quad (7)$$

where, P_h is the hydraulic power (kW), Q is peak runoff rate (m^3/s), ρ is density of fluid (1000 kg/m^3), g is acceleration of gravity (9.81 m/s^2), H is differential hydraulic head (m), and η is the efficiency coefficient accounting for the hydraulic head loss. $H_{100\%}$ is the average differential head calculated by assuming 100% hydraulic efficiency (η = 1). - no head loss - based on the mean ground slope (S) multiplied by the maximum travel distance (L) of the surface runoff within each drainage. $H_{100\%}$ is evaluated solely based on the hydrologic drainage parameters (i.e., S and L) evaluated in Table 4, Table 6, and Table 8. Therefore, neither effect of the ground concaveness nor head loss due to surface roughness or porousness was accounted in the present $H_{100\%}$. Table A- 2, Table A- 4, and Table A- 6 present the converted values for P_h in the unit of kilowatts (kW) and horsepower (hp), respectively, after applying the conversion factor 1.341 to P_h in kW.

4. DESIGN OPTIONS FOR STRUCTURAL BEST MANAGEMENT PRACTICE (BMPS)

A stormwater best management practice (BMP) is a technique, measure or structural control that is used for a given set of conditions to manage the quantity and improve the quality of stormwater runoff in the most cost-effective manner (US EPA, 1999). Low impact development (LID) is a comprehensive approach to land development (or re-development) that works with nature to manage stormwater as close to its source as possible through strategically integrated stormwater controls (BMPs) distributed throughout the landscape. The primary goal of LID is to recreate the predevelopment site hydrology through site design techniques that promote storage, infiltration, evaporation, and treatment of runoff. LID employs principles such as preserving and recreating natural landscape features, minimizing effective imperviousness to create functional and appealing site drainage that treat stormwater as a resource rather than a waste product (US EPA, 2014).

This chapter provides a review of scientific, technical, and policy documents pertaining to mitigation strategies for rainwater runoff issues plaguing coastal communities nationwide. The overall goal of this chapter is to provide local decision makers with the knowledge and resources to apply LID practices to the community, neighborhood, and site scale by presenting essential information that should be considered when deciding on stormwater management practices for a specific site. The urban stormwater BMPs shall be designed to reduce, redirect, or delay the runoff flows so that the post-development peak discharge rate, volume and pollutant loadings to the receiving water can be effectively mitigated and managed. A number of sometimes competing factors need to be addressed. Therefore, the present review particularly concerns the BMP designs that can be effectively integrated into the existing/future site development condition of Galveston Island beaches, aesthetically pleasing, and can accommodate and enhance the biodiversity and local beachgoers safety. For this purpose, the stormwater BMP options are grouped based on their primary functions against the urban rainfall runoff that address the prescribed design concerns and presented in Table 2.

The review of each BMP design option is structured to include general descriptions on the design principles, technical feasibility (site applicability), and important components to incorporate into the design, as well as advantages and limitation of each stormwater management practice. Reference material for each BMP design option is cited in the *Reference* chapter. Only unique information presented by a certain publication was cited directly within the text.

Table 10. Stormwater BMP categories based on primary stormwater management function.

Stormwater Management Function	**BMP Design**
Infiltration systems	
	Infiltration basins
	Infiltration trenches
Pervious pavement systems	
	Pervious asphalt, concrete, paver block, reinforced turf
Detention systems	
	Dry detention basins (dry ponds)
	Subsurface detention systems (underground vaults)
Retention systems	
	Retention basins (wet ponds)
	Retention berms (retentive grading)
Ecological engineering systems	
	Dune infiltration systems
	Bioinfiltration/bioretention
Bio-infiltration systems	
	Vegetated swales
	Vegetated filter (vegetated buffer) strips
Passive beach dewatering systems	
	Gravity drainage system (strip drainage system)
	Beach drainage systems (toepassing drainage systems)
	Pressure equalizing modules (vertical drainage systems)
Armoring and diversion systems	
	Rock and timber revetments
	Gabion revetments
	Bio-thatching

4.1 Infiltration Systems

Infiltration systems are shallow, impounded areas, typically filled with stone or an engineered soil mix. Stormwater infiltration systems are designed to intercept and temporarily store some volume of the surface runoff in the filling media over a duration of several hours or days (up to 72 hours, (US EPA, 1997)) until they are allowed to infiltrate into the underlying substrate. These systems provide reduction in runoff rates, volumes, and pollutant loads by promoting the return of the captured surface runoff into the hydrologic cycles through infiltration into subsoils. The size and structural design can vary from one large basin to multiple, smaller impoundments or strips (i.e., infiltration trenches) depending on the drainage conditions. Two most common types of the infiltration systems are surface infiltration basins and infiltration trenches.

4.1.1 *Infiltration Basins*

The basin system is a shallow, landscape excavation filled with engineered substrate to promote the hydrologic processes (evaporation and transpiration) of the permeated runoff before being infiltrated into the nearby substrate. In the substrate a layer of stone or soil used as the underlying base for a BMP (shown in Figure 5). The basin system is designed to only intercept a certain volume of runoff, thereby any excess volume will be bypassed. The system is not designed to retain a permanent pool volume but rather to transform the surface water into a groundwater flow and to remove pollutants through filtration, adsorption, and biological conversion. The infiltration basin should be used on drainage areas up to 50 acres designed to drain within 72 hours in order to receive runoff from the next event (US EPA, 1993a).

Design Principles

Figure 4 shows the layouts of the typical infiltration system. Figure 5 illustrates the profiles of the typical infiltration system and composition of the infiltration layer. The size of the infiltration basin shall be designed primarily for frequent storm events (i.e., 2-yr or 1.5", or 38 mm, storm) so it can drain without overflow above the infiltration layer. The maximum allowable ratio of the contributing drainage area (CDA) to the design footprint is 5:1 (Pennsylvania Department of Environmental Protection, 2006). However, it should also be able to convey and mitigate the peak flow of more intense storms (such as the 100-yr, 24-hr duration) if the positive overflow structures (e.g., emergency spillway) of at least the 10-year, 24-hr storm conveyance are provided. The positive overflow structure is a drainage structure that conveys the excess flow as an overland flow along an open course (City of New Braunfels, 2016). At least 1 ft (0.4 m) of freeboard above the 100-yr stormwater elevation should be maintained and proper overflow or discharge from the infiltration basin should be considered in the design. For large basins, multiple outlet control devices are required. The design should allow a minimum 2-ft buffer between the infiltration bed (or bedrock) and seasonal high groundwater table. The underlying native soils must be permeable with infiltration rates higher than 0.3 in/hr or 25.4 mm/hr (i.e., hydrologic soil group A and B soils) and the distance from the bottom of the infiltration system to the seasonal high water table must be at least 0.5 feet (Blick et al., 2004; Ellis et al., 2014). The infiltration basin is not an underdrained system but an optional (backup) underdrain or overflow structure can be installed to prevent the standing water problem within the basin and for extreme flood control for larger storms. Underdrains are typically perforated pipes in stone layers or trenches that intercept, collect, and convey stormwater that has percolated through soil, a suitable aggregate, and/or geotextile, in order to drain the BMP after a storm event (Philadelphia Water, 2018). The layout of the underdrain system is shown in Figure 5. Care should be given to allow as little compaction as possible of the underlain soils but the berms surrounding the basin should be compacted earth with a slope of not steeper than 1/3 and a top width of at least 2 feet. The inlets into the basin should have erosion protection and may have a sediment trap or water quality insert to prevent large particles from clogging the system. Additionally, adequate pretreatment (e.g., vegetated filter strip or sediment forebay) must be provided upstream of the infiltration system to prevent sediment from reaching the inlet of the infiltration basin and causing it to clog and fail.

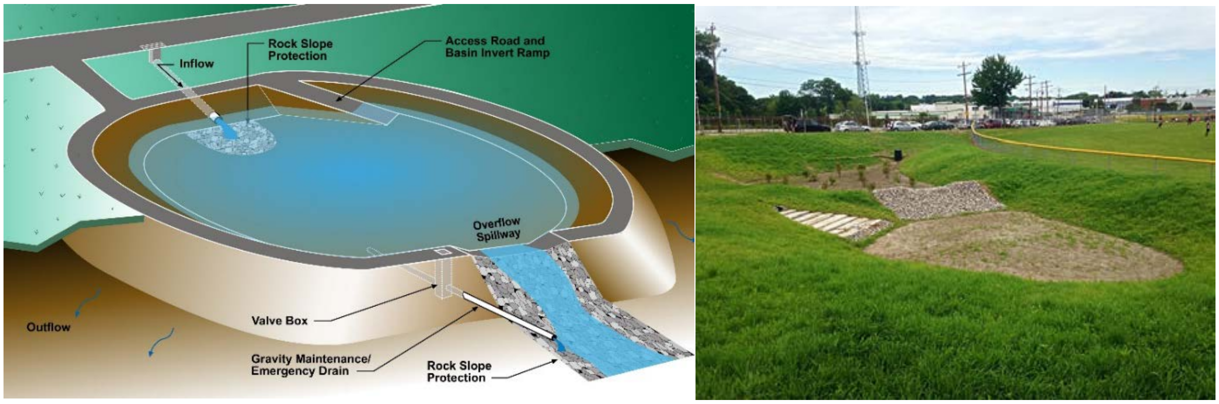

Figure 12. Schematic (left) and photo (right) of typical infiltration basin system layouts (Illustration from keneulie.wordpress.com, Photo from epa.gov).

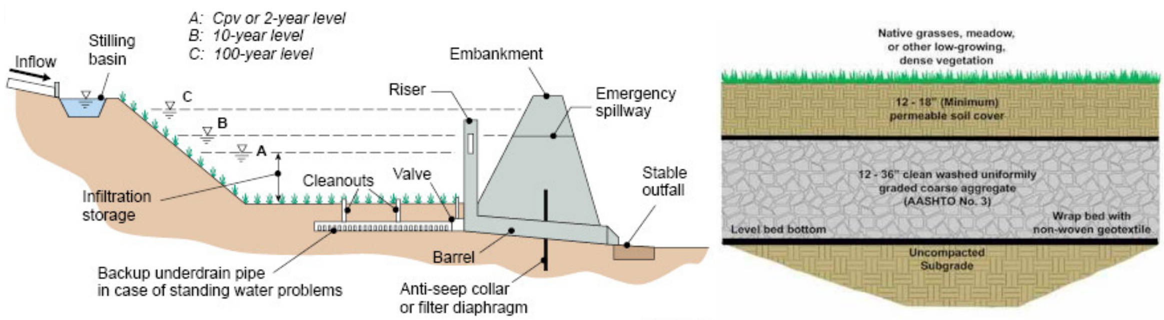

Figure 13. Schematics of typical infiltration basin profile (left) and infiltration layer (bottom). Source: Maryland Department of the Environment, 2000 (left), and Pennsylvania Department of Environmental Protection, 2006 (bottom).

Benefits & Suitability

The basin system can be integrated into a local development plan to provide both water quantity and quality controls, as water percolates through the various substrate layers (Figure 13), and attractive landscaping features. The infiltration system also provides additional benefits by increasing recharge of underlying aquifers. This recharged water serves to provide baseflow to streams and maintains runoff water quality (Pennsylvania Department of Environmental Protection, 2006). However, poor design, poor construction, or neglected maintenance can cause failure of the system especially when implemented on a development site with high sediment loads. The key to promoting infiltration is to provide enough surface area for the volume of runoff to be absorbed. The infiltration system is inappropriate at sites where groundwater is the primary source of drinking water, especially nearby commercial or industrial areas where the potential for contamination (organic pollutants or metals) migration through runoff is high. The excessive sediment accumulation or compaction of the underlying soil layers during construction can reduce the infiltrative capacity or cause clogging of the system. Therefore, the system should not be placed on sites with disturbed substrate (due to recent construction activity or site grading within 5 years) and frequent maintenance is required. Root systems of dense vegetation or grasses planted on the drainage surface can help stabilize and increase the permeability of soils while preventing migration of pollutants through adsorption and biological conversion.

4.1.2 Infiltration Trenches

Design Principles

Infiltration trenches (or wells) are underground (subsurface) infiltration systems that place storage media of varying types beneath the proposed surface grade. Trench systems can come in a variety of configurations but commonly the excavation is filled with stone beds consisting of clean-washed, graded aggregate wrapped with geotextile in order to temporarily store stormwater before letting it infiltrate into the underlying or surrounding native substrate. Figure 6 shows a typical infiltration trench system. As the possibility of groundwater contamination exists, the proximity to a source of drinking water needs to be considered before implementation. The trench system is primarily designed to capture only small amounts, the first flush, of runoff to control the peak flow and can be used for drainage areas up to 2 acres where sediment loads are relatively low. For enhanced storage and infiltration capacity, underground pipe and chamber storage comprised of perforated pipes or pipe-like linear chambers, or underground plastic grid storage consisting of inter-connected and stacked plastic structures can be placed beneath the stone beds. Topsoils should be place over the stone bed with a minimum of 6″ depth when the perforated pipes are underlain (i.e., leaky piping system) and vegetated for stabilization. The stormwater conveyed into the subsurface storage media can be distributed via a network of perforated pipe systems. It is critical to contain at least 40% void volume in the aggregate stone media. The slope of the infiltration bed bottom should be level or with a slope no greater than 1% in order to ensure even water distribution and infiltration. All subsurface infiltration systems should be designed to include positive overflow in the outlet control structure for extreme storm events. Generally, the top of the subsurface storage pipes should be at least 4 inches deeper than the top of the aggregate.

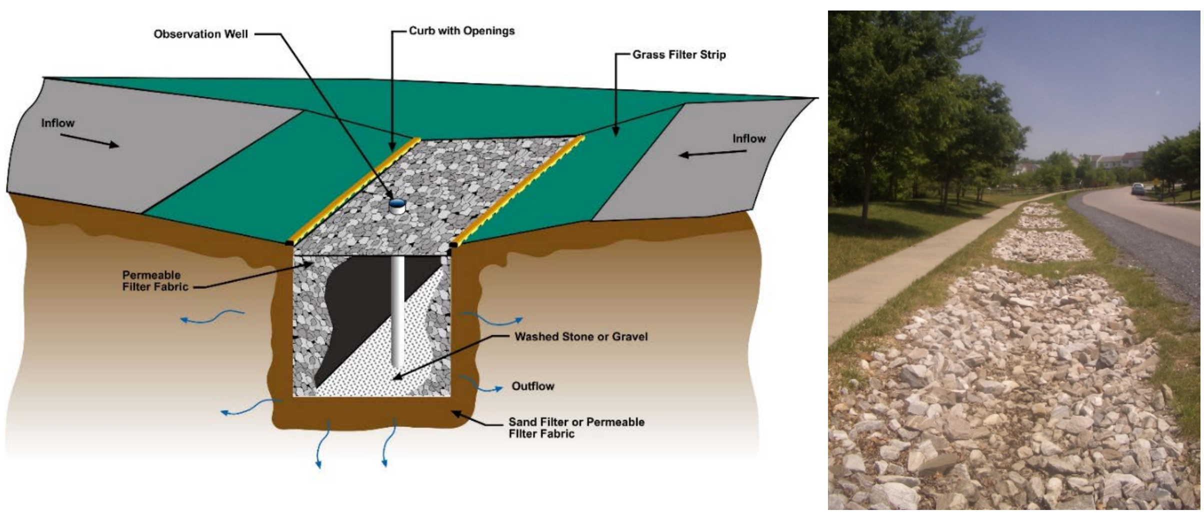

Figure 14. Schematic (left) and photo (right) of typical infiltration trench system (Illustration from keneulie.wordpress.com, Photo from sustainablestormwater.org).

Benefits & Suitability

Trench systems can be a stand-alone feature ideally suited for expansive, generally flat open spaces but can be stepped or terraced on downsloped terrains as long as the base of the system remains level. The subsurface infiltration system is a hidden feature buried underground and hence provides a flexible design option beneath lawns, recreational areas, parking lots, and other impervious open areas, overcoming the space constraints and utilizing otherwise undevelopable land. Roof runoff, catch basins, and other area drain inlets can be directly connected to the subsurface infiltration system (i.e., dry wells, seepage pits) but sediment traps or sumps are recommended between the invert of the discharge pipes and the infiltration bed. The "invert" refers to the lowest point on the inside of a sewer or other conduit. The system installation can be more costly compared to the surface infiltration systems and contingency plans for regularly scheduled inspection and maintenance are required.

4.1.3 Pervious Pavement with Infiltration Bed

Pervious pavement facilitates stormwater infiltration and provides temporary storage but primarily allows stormwater to pass through small voids of the stone bed for peak rate control.

Design Principles

Figure 15 shows the schematic (left) and photos (middle and right) of pervious pavement systems. Pervious pavement consists of a permeable surface layer underlain by a uniformly-graded stone bed. The surface pavement may consist of pervious asphalt, pervious concrete, or pervious pavement units. The underlain stone bed can be made with uniformly graded and clean-washed coarse aggregate, 1-1/2 to 2-1/2 inches (38-64 mm) in size, with a void space of at least 40%. A layer of geotextile filter fabric separating the permeable surface course from the underlying, uncompacted soil mantle prevents the migration of fines into the bed bottom. Water within the subsurface stone bed should always be kept below the level of the pavement surface and the bottom of the pavement system should be laid at least 0.5 feet higher than the seasonal high water table in order to prevent overflow. If the infiltration rate through the subgrade bottom soil does not exceed 0.3 in/hr or 7.6 mm/hr, the underdrain system and permeable pavement should be installed in the subgrade in order to completely drain within 48 hours. The subgrade is a layer of stone or soil used as the underlying base for a BMP (i.e., pervious surface pavement). The permeable surface slope must be less than 5% to ensure the stormwater storage capacity and the ratio of permeable surface to surrounding impervious drainage area should not exceed 1:5. Figure 16 exemplifies the application of the pervious pavement system for the construction of a jogging path.

Benefits & Suitability

Pervious pavement with underlying infiltration systems help reduce both the rate and volume of runoff and recharge the groundwater. The system can also provide measurable contaminant reduction in the pollutant load of runoff by filtering through voids. Properly installed and maintained pervious pavement can properly function for a life-span of over twenty years (Ellis et al., 2014). Since water drains through the surface course and into the subsurface bed, the pervious pavement tends to be less affected by freeze-thaw cycles and provides better traction for walking paths in rain or snow conditions. Pervious pavement systems can be used in paths and driveways that would otherwise be covered by the standard pavement without consuming valuable land. The system is particularly well suited for use on urban development sites and in low (occasionally heavily loaded) traffic areas. Withstanding strength against compression is low, therefore use for roadways or highways has been limitedly to providing lateral surface drainage. Pervious pavement materials are generally 10% to 20% higher in cost and installation should be performed only by trained personnel.

<u>Applications</u>

Pervious pavement comes in a variety of forms. Variations include, for example, pervious asphalt, pervious concrete, pervious paver block, reinforced turf (gravel filled grids), etc. While pervious asphalt is very similar in appearance to standard asphalt, pervious concrete has a coarser appearance than conventional concrete. Care must be taken to avoid creating an impervious surface layer during the construction process due to the compression overload. Reinforced turf applications are excellent for overflow parking, playground, pathways, and driveways to reduce the impervious surface area.

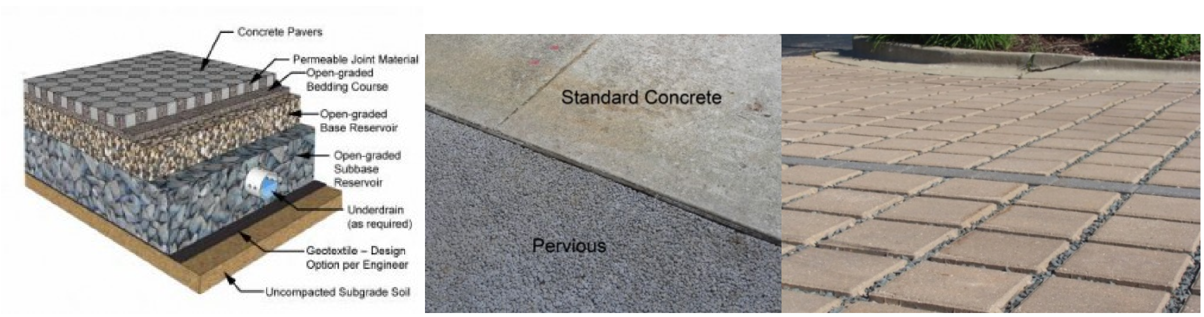

Figure 15. Schematic (left) and photos (middle and right) of pervious pavement systems. A typical layer structure of the pervious pavement system with an underdrain system (left), a pervious concrete surface compared to the standard counterpart (middle), and interlocking pervious pavers (pervious blocks) installed in a neighborhood pathway (right, Source: Interlocking Concrete Pavement Institute).


Figure 16. Photos of construction phases of a reinforced turf application for a jogging path (Source: Invisible Structrures, Inc.).

4.2 Detention Systems

Detention systems intercept and temporarily retain a certain volume of stormwater runoff for subsequent, gradual release to a receiving stream or sewer system. Detention systems do not retain a significant permanent pool of runoff water but are designed to completely empty out between runoff events (i.e., within a period of less than 24 hours). Detention systems are designed to optimize the detention time, delay the peak discharge, and provide mainly runoff quantity control as opposed to water quality control. The detention time is defined as the time from when the maximum storage volume is achieved until the time when only 10 percent of that volume remains in the basin (*Pennsylvania Stormwater Best Management Practices Manual*, 2006). Standard detention systems can provide limited settling of particulate matter by gravity, but a large portion of this material can be re-suspended by subsequent runoff events. Typical features of detention facilities used to manage stormwater runoff include dry detention basins and underground vaults, pipes or tanks.

4.2.1 Dry Detention Basins (Dry Ponds)

Dry detention basins, also called “dry ponds”, provide surface water impoundment in a natural depression or excavation of existing substrate yielding temporary storage for runoff. These systems are designed to completely drain within two to three days after the rain stops and function hydraulically to attenuate stormwater runoff peaks.

<u>Design Principles</u>

Dry detention basins utilize inflow, pretreatment, low-flow channel, temporary ponding, embankment, and outlet structures. Figure 16 (top) shows schematic of the typical dry detention basin design. They are designed to control overbank flooding (5-yr through 25-yr design storm) and downstream bank erosion (2-yr peak design flow rate) but can be designed to control the extreme flood (100-yr) storm event as well. Dry detention systems should be designed for retaining the sufficient volume (2-yr to 100-yr design storm) and typically require a footprint of 1% to 3% of their CDA. The CDA to the dry detention basin is determined based on annual rainfall, local soil permeability, and outlet sizing, and drainage areas greater than 10 to 25 acres are recommended. These basins usually have a minimum width of 10 feet and a minimum length-to-width ratio of 2:1 to maximize sedimentation. The basins are designed to maximize the length of stormwater flow pathways and irregularly shaped basins are recommended to appears more natural. Dry detention basins must have a primary outlet structure (e.g., single-stage or multi-stage outlets) that regulates the flow and promotes the settlement of pollutants. A secondary outlet (or spillway) is needed to convey the release of the maximum runoff discharge for the 100-year storm event (DNR, 2008). The primary outlet structure controls runoff peak rates for required design storms and incorporates a combination of weirs, orifices, pipes, and energy dissipaters at the end of the outlet to prevent erosion. Outlet trash racks should be installed so that debris will be lifted by higher flows.

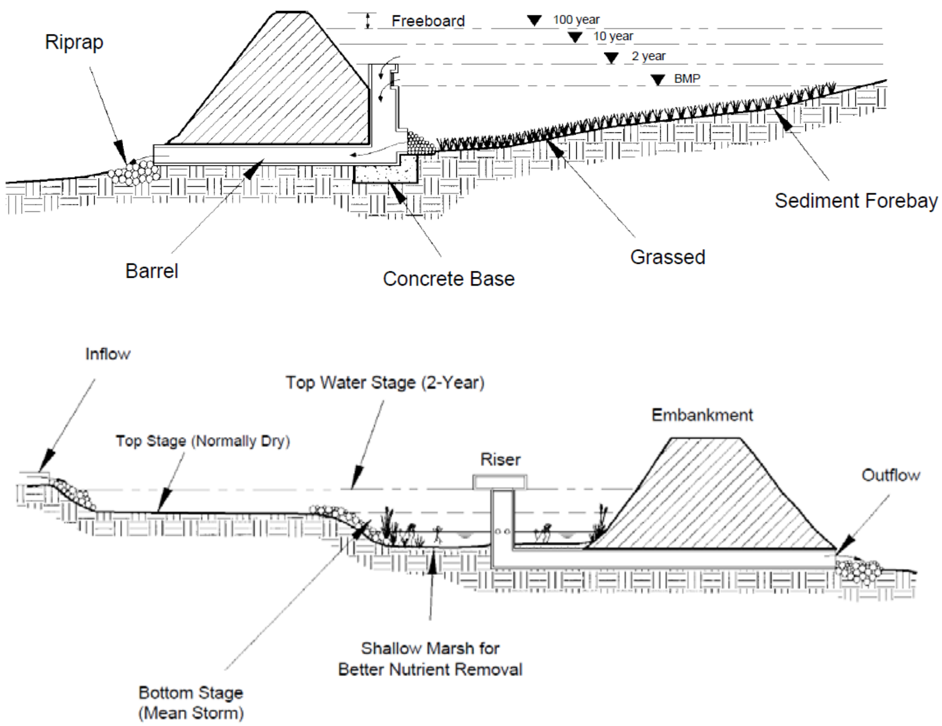

Figure 17. Schematic cross-sections of typical detention basin designs. Dry pond (top) and extended detention basin (bottom). (Source: Northern Virginia BMP Handbook, 1992).

<u>Benefits & Suitability</u>

If properly designed and managed, dry detention basins can be a cost-effective, authentically plausible practice providing controls for peak rates of stormwater discharge to downstream areas, thereby reducing the effective shear stress on the bed and banks of the receiving bodies of water. Some runoff volume reduction can be achieved through initial saturation of the soil mantle and evaporation that takes place during detention but the net volume reduction for design storms is minimal. Water quality benefits are limited and occur through settlement of the larger particulate fraction of suspended solids.

Where feasible, dry detention basin systems can be extended to multi-stage basins in order to increase detention capacity beyond that required for stormwater peak rate control and maximize sedimentation for enhanced water quality benefits. The extended detention basins utilize a combination of smaller permanent pools of water of 4-8 feet deep (e.g., forebay, micropool) near the inlet and outlet points and temporary extended water storage above the permanent pool with some elements of shallow marshes or wetlands of 0-9 inches depth. The permanent pools serve to protect the sediment settlement from being re-suspended and additional pollutant removals can be achieved by algal uptake from the ponding water and wetland vegetation. Figure 16 (bottom) shows the schematic of the extended dry detention basin and Figure 17 shows an example of the extended detention basin incorporating the micropool.

Dry detention basins are widely applicable for most land uses but require a relatively large space as they are best suited to drainage areas greater than 10 acres. The small outlet size designed for drainage areas of smaller size will require increased maintenance due to frequent sediment clogging. The standard dry detention basin provides only marginal removal of pollutants and is not appropriate for ultra-urban areas, or an on-line location within a stream network. Poorly maintained basins can create nuisance odors, weed growth and accumulation of trash.

Figure 18. Photo showing an example of an extended detention basin incorporating a micropool and vegetation planting. (Source: http://www.matternandcraig.com).

4.2.2 Subsurface Detention Systems (Underground Vaults)

Subsurface detention systems comprise of underground stormwater storage such as vaults, pipes and tanks that are often used in conjunction with other stormwater management designs for runoff quantity control, particularly for space-limited areas. Stormwater captured by a riser pipe connected to the catch basin or curb inlet flows into a series of chambers or storage compartments. The captured runoff is retained throughout the storm event and may be released directly into surface water through an outlet pipe.

<u>Design Principles</u>

Subsurface detention systems can take on a variety of forms. Figure 18 shows some examples of typical subsurface storage systems. Typically, subsurface detention systems are built with buried concrete or stone beds rapped in geotextile, perforated plastic/metal pipes, or stacked and inter-connected plastic (e.g., high density polyethylene) structures. The perforated pipe or chamber storage structures are often placed in a stone bed to increase the stormwater detention capacity. As much as 95% permeability, that is the void surface per unit area between sediment grains, can be achieved by using plastic grid storage modules but with significantly higher installation cost. A pretreatment system consisting of a sediment sump or vault chamber needs to be provided at the inlet to remove sediment and debris before discharge to a subsurface storage tank and needs to be sized to capture 0.1 inches of runoff (Ellis et al., 2014). The sediments accumulating in the stone or grid storage systems can compromise subsurface detention systems and therefore these systems require strictly scheduled regular inspection and maintenance. Inspections are required every six months and within 24 hours after every storm event greater than 1.0 inches to clean out accumulation of oil and sediment. Outlet controls are used to regulate the rate of discharge from the subsurface storage and maintain a design water surface elevation during various storm events. They also provide a measure for bypassing the flows from large storm events. Underground detention structures need to be designed for potential overburden support and traffic loading and all construction joints must be watertight.

Benefits & Suitability

Subsurface detention systems can be a good option to control peak flow rates of stormwater runoff for high density or urban areas with strict space constraints. Subsurface detention designs allow for easily adaptable footprints that can fit into almost any size space and can be easily incorporated into other surface stormwater management practices as a part of the overall development plan. The system can be placed beneath lawns, recreational areas, parking lots, buildings, or other impervious areas when space constraints exist. It is a hidden system buried underground without occupying surface or rooftop space and hence, aesthetically adverse effects are minimal. Although some systems are designed to allow infiltration to recharge groundwater (i.e., underground retention), typical subsurface detention systems provide little or no water quality improvement nor net volume reduction. Addition of pretreatment features, such as forebay or pre-settling chambers at the system's inlet can facilitate improvements to water quality by removing floatables and trapping some level of sediments through deposition. The installation requires extensive and costly excavation and material cost and maintenance cost are more expensive compared to other surface stormwater BMPs.

4.3 Retention Systems

Retention is, according to the strict definition, a practice providing storage of stormwater runoff without subsequent surface discharge (WEF/ASCE, 1992). This means water volume reduction can be realized only by either infiltration or evaporation. In stormwater management, retention systems are designed to capture a volume of runoff and retain that volume until it is displaced in part or in total by the next runoff event. Retention systems can provide both water quantity and quality control. There are variations of retention systems and often, if coupled with infiltration systems, the retention systems can function to aid additional runoff filtration and local groundwater recharge. Retention basins and retentive grading described herein are the most common and effective types of retention systems.

Figure 19. Schematic and photos of typical subsurface stormwater detention systems. A diagram of subsurface detention system (top-left, source: Montgomery County, MD) and plastic chambers (top-right, source: Island Health), concrete chambers (bottom-left, Fairfax County, VA), and plastic grid modules (bottom-right, source: www.varitech.com).

4.3.1 Retention Basins (Wet Ponds)

Retention basins or wet ponds are stormwater basins that include a substantial permanent pool for water quality treatment and additional capacity above the permanent pool storage for temporary runoff storage. Water in the pond above the permanent pool level is displaced in part or completely by the runoff volume from subsequent runoff events. Pollutant removal in retention ponds can occur through a number of mechanisms. The main mechanism is sedimentation, the removal of suspended solids and associated pollutants through gravity settling. The presence of a permanent pool of water can also prevent the sediments accumulated in the pond from being suspended and washed out. Aquatic plants and microorganisms inhabiting the ponding area can provide uptake of nutrients and degradation of organic contaminants. Retention basins incorporate an aquatic bench (a dense stand of emergent wetland vegetation) around the perimeter of the pond and provide added pollutant (metals and nutrients) removal efficiency through filtration by aquatic plants.

Design Principles

Figure 19 shows a typical layout of retention basins. Retention basin systems should include one or more forebays that trap coarse sediment at all major inflow points. The forebays should be built with 4 to 6 feet of depth and have a capacity to contain 10 to 15 percent of the total permanent pool volume. The forebays should be physically separated from the rest of the pond by a berm, gabion wall, etc. The permanent pool should be designed for a maximum depth of less than 8 feet, and the extended storage depth of at least half an inches on the top. Pond perimeters should be covered by a dense stand of emergent wetland vegetation that are ideally tolerant of a range of depths, inundation periods, and non-invasive, perennial plants that establish quickly. A water surface elevation (level) of the permanent pool should be maintained during all wet periods in order to avoid adding stresses on vegetation covering and surrounding the ponding area. A wet pond system requires a footprint of generally 1 to 3 percent of CDA, and is suitable for drainage areas of at least 10 – 25 acres. A means of sustaining a constant inflow should be implemented to ensure the water quality for smaller drainage applications and to improve the biological health and effectiveness of the ponding system. A length to width ratio of at least 2:1 and a bottom slope of 5:1 (H:V) or flatter are recommended for basin configuration. Generally, hydrologic soil groups "C" and "D" (low to poor permeability) are suitable for ponding areas without modification, but organic soils should be used for vegetation planting areas. Vegetation is an integral part of wet detention pond systems and wedge-shaped, varying-depth basin configurations can promote vegetation growth in the shallow areas. Outlet devices are generally multistage structures with pipes, orifices, or weirs for flow control installed in the embankment for easy access. A pond drainage system should also be included that can completely drain the permanent pool within 24 hours for maintenance. An emergency spillway should be installed so it can safely convey 100-yr storm flows.

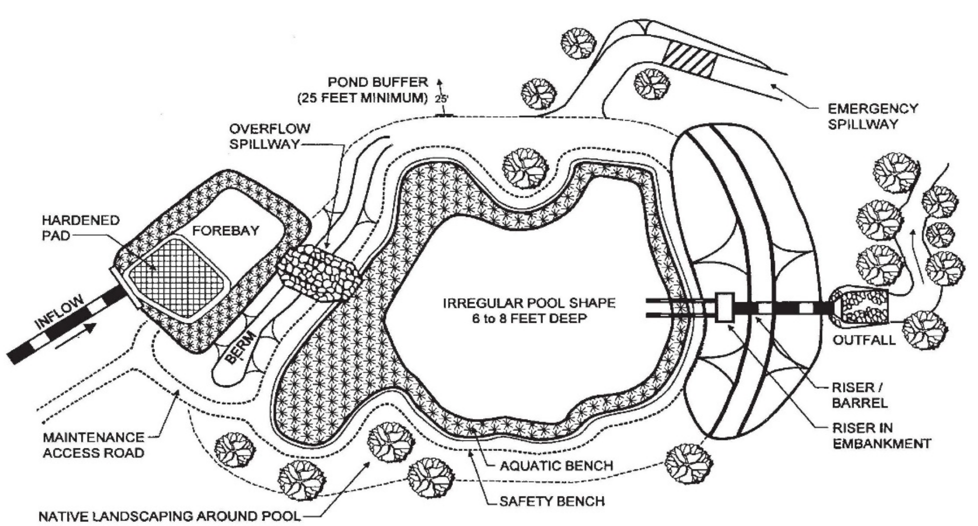

Figure 20. Schematic of a typical layout of retention basins (Source: Maryland Department of the Environment, 2000).

Benefits & Suitability

Figure 20 shows some examples of retention basins. Wet detention systems can be effective for pollutant removal and peak rate mitigation and provide aesthetic and wildlife benefits. Due to the potential to discharge warm water, wet ponds should be used with caution near temperature-sensitive waterbodies (e.g., cold water streams). Properly designed and maintained, wet ponds generally do not support significant mosquito populations. Costs for construction and space demands per required drainage volume are low relative to other stormwater management practices. Soils excavated for the pond construction can be used for filling and other landscaping as needed for construction of low-lying coastal areas. Small ponds or under-sized ponds can reduce the aesthetic attractiveness and cause mosquito breeding and unpleasant odor and are thus not appropriate for drainage areas less than 10 acres. The wet detention systems cannot be placed on steep unstable slopes and a measure to maintain the surface water level of the permanent pool should be incorporated to prevent repetitive wetting and drying of the substrate as this can disturb plant establishment.

Figure 21. Photos of retention basins installed alongside a road (left, source: City of High Point, NC) and alongside a residential area (right, source: Charles County government, MD).

4.3.2 Retention Berms (Retentive Grading)

Berms and other retentive grading options comprise of a linear feature created by filling or excavation of an upslope area intended to create a barrier for flows and slow down, retain and promote infiltration for volume control and stormwater diversion.

<u>Design Principles</u>

Figure 21 shows a schematic cross-section of a retention berm installed alongside a roadway. Berms are constructed earthen embankments with sloping sides that are usually placed parallel to an existing site's contours. Retention berms are mounds of sand or stones covered with soil and vegetation that collect and temporarily store stormwater runoff, allowing it to infiltrate into the ground and recharge groundwater. Of particular, retentive grading (i.e., diversion berms) are compacted earth ridges usually constructed across a slope in series to intercept and direct stormwater flow in order to promote longer flow pathways, thus increasing the time of concentration. Retentive grading can be used to protect slopes from erosion and to slow runoff velocity while providing greater opportunity for pollutant removal and infiltration.

Berms or ridges for retentive grading can be constructed in series downside of the slope to retain and spread large quantities of runoff along multiple levels. In some cases, retentive grading may be created with original (site) grade by excavation or removal of upslope materials, effectively creating shallow depressions (i.e., infiltration berms, Figure 22). The level to which the berms can provide runoff rate and volume controls can be limited depending on design configuration (berm crest height, retentive slope, etc.), soil permeability, canopy cover, and slopes of the development site. A maximum ratio of 5:1 between the CDA to the infiltration area covered by the berms is recommended and the use of uncompacted, permeable soils is appropriate. A low berm height not exceeding 2 feet is recommended to prevent excessive ponding. Higher berm heights may be used in order to divert flows into the direction parallel to the contours and hence lengthening flow pathways toward a nearby channel, facility, or receiving body of water. Generally, more berms of smaller size are preferable to fewer berms of large size as such a series of berms can serve more effectively for infiltration and stabilization of the slope. It is recommended that berms be installed to be level across the contours with a ratio of 3:1 (H:V) of the side slope. Berms can be built purely with high-quality topsoil but the inner portion of the berms beneath 4 – 9 inches of topsoil can consist of well-draining, stable fill materials to reduce cost and promote stabilization. Planting native trees and shrubbery is recommended as their deep root systems can prevent erosion and promote berm stabilization. Site conditions for soil, hydrology, and light shall be considered when choosing the plant species.

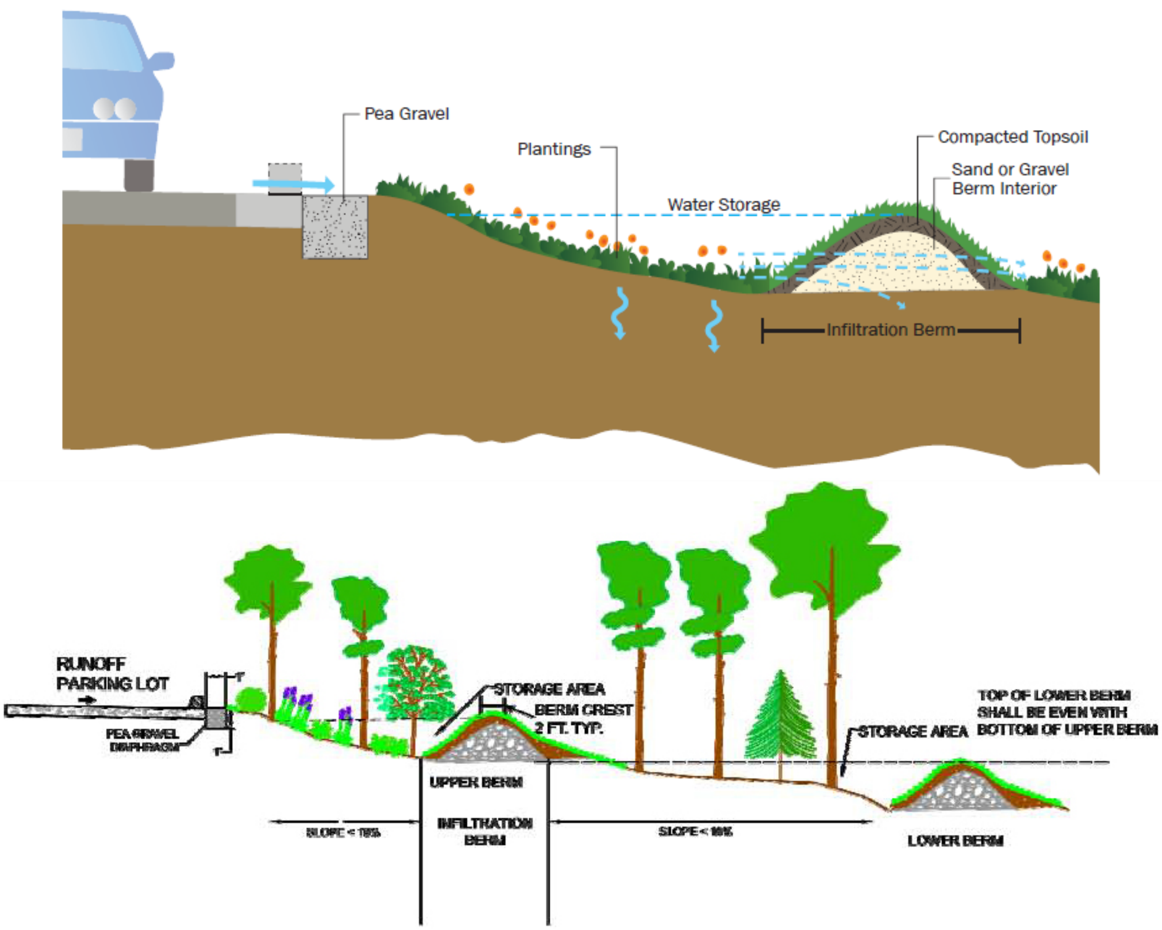

Figure 22. Schematic cross-sections of a retention berm and a retentive grading installed alongside a roadway (left, source: Anne Arundel County, MD) and alongside a parking lot (right, (Maryland Department of the Environment, 2000).

Benefits & Suitability

Berms and retentive grading primarily provide runoff rate and volume control but some retentive grading created by shallow depressions (Figure 22) can also promote infiltration of runoff and groundwater recharge. Berms and retentive grading are ideal mitigation measures for relatively small impervious areas to intercept runoff from roadways, parking lots, or sloping terrain where there is less than a 10% slope in topography. Berms and retentive grading cannot be constructed on the sites with nearly flat terrain (slopes less than 1%) or with slopes where soils have low shear strength (i.e., known as landslide prone area).

Berms and retentive grading techniques can serve multifunctional purposes and are easily incorporated into the landscape. Berms are often used in conjunction with existing recreational features, such as pathways and wooded hillsides, and retentive grading can be employed to provide infiltration capacity for a specific part of feasible sites. They may function alone in vegetated areas or may be incorporated into the design of other stormwater control facilities, for example, for pretreatment of diffuse sheet flow and on the downslope side of an infiltration basin to provide infiltration and detention while stabilizing the slope. Berms and retentive grading are suitable for a wide variety of drainage settings, both large and small, and urban areas including parking, commercial and light industrial facilities, roads and highways, residential developments, and vacant lots. Aside from the stormwater management purpose, berms and retentive grading can also serve as noise barrier, separation and screening of undesirable views and conflicting uses, and forming more interesting landscaping against plain terrains. They are cost effective stormwater management measures that can be built by utilizing available on-site soil. Due to the limited capability of controlling the runoff volume, berms and retentive grading are not appropriate to control runoff from very large, highly impervious sites. They may be impractical for ultra-urban drainage due to the space constraint inherent in the requirement for vegetation coverage. Berms and retentive grading should resemble the surrounding natural landscape to appear aesthetically pleasing.

Figure 23. Schematic and photo of examples of berms and retentive grading in urban settings. A street side rain garden concept implementing diversion berms created by shallow depressions on the upslope (left image, source: Pinterest) and retentive grading in a residential area in Philadelphia (right image, source: Philadelphia Water, 2015).

4.4 Ecological Engineering

Ecological engineering is defined as the design of sustainable ecosystems that integrate human society with its natural environment for the benefit of both and combine system ecology with the process of engineering design (Mitsch and Joergensen, 1989). It is becoming a broadly recognized paradigm to utilize natural energy sources as the predominant input to manipulate and control environmental systems. In many stormwater BMPs for LID, a group of BMPs are utilized in series, so-called “treatment train”, in order to maximize the opportunities for runoff flows from one to the next to be treated for both runoff reduction and pollutant removal, and provide greater flexibility in the stormwater management designs (Ellis et al., 2014). While many different combinations of treatment trains are possible, here most prominent ecological engineering practices of dune infiltration systems and bioinfiltration / bioretention that incorporate infiltration, retention / detention, and filtration mechanisms are reviewed.

4.4.1 Dune Infiltration Systems

Dune infiltration systems (DIS) are a variation of subsurface infiltration systems coupled with retentive berm technology. DIS has been first developed for implementation on the coastal beaches in North Carolina in order to help reduce direct stormwater discharge and prevent polluted surface runoff from reaching the beach and ocean without treatment. The DIS is designed to capture stormwater runoff and promote infiltration and groundwater recharge.

<u>Design Principles</u>

The DIS unit consists of an underground chamber, diversion vault, distributing pipes, bypassing outlet, and soil fills burying the DIS within the ground excavation. Figure 23 shows layout and photo of a North Carolina dune infiltration systems. A portion of stormwater discharge from existing discharge pipes or stormwater drop points are diverted and conveyed into open-bottom chambers located beneath the sand dunes. Stormwater reaching the chambers spreads laterally and infiltrates into the sand. Pollutant removal can be achieved by natural biophysical processes as the surface runoff infiltrates into the ground, is mixed with groundwater, diluted, filtered between soil particles, and decomposed due to environmental stresses and micro organic mechanisms while traveling along the underground downslopes. During extremely intense rainfall events, stormwater exceeding the DIS capacity is allowed to bypass the system and discharge to the ocean through the existing discharge pipe.

The ideal site for the DIS was reported to be an elevated dune system with an annual mean water table several feet below the surface. No adverse impact on dune stability or groundwater systems were generated when the DIS is implemented for low stormwater drainage areas less than 10 – 15 acres permeating substrate capable of handling more than 50 inches of water infiltration per hour is recommended. Open-bottom, high-density polyethylene chambers can be placed beneath the dunes in various arrangements. Ditches are excavated down to a target elevation so the existing stormwater discharge inlet can be positioned upslope of the dunes. The chambers are placed in a layer of gravel poured on the bottom of the excavation in order to promote infiltration. This layer should be protected by geotextile fabric from intrusion of surrounding sand. Top soils should cover the chamber with a minimum of 1.5 feet depth and native vegetation should be planted to aid in dune stabilization.

Figure 24. Layout and photo of a North Carolina dune infiltration systems. A general layout of the dune infiltration systems (left) and a photo of open-bottom chambers for underground storage (right, source: NC State Extension available at www.ces.ncsu.edu).

Benefits & Suitability

The DIS is a relatively new stormwater management technology first publicized in 2011 (Bright et al., 2011). After 3 years of post-installation monitoring, Price et al. (2013) reported that the DIS has helped intercept up to 97% of cumulated volumes of stormwater inflows, temporarily increased the ground water table for up to 2 weeks, and significantly reduced bacteria levels (more than 60% reduction in the measured, single-sample maximum concentration of enterococci). Due to the short history of implementation, there is no standardized design guideline for universal application, yet. Therefore, watershed assessments and local soil surveys should be performed by engineers to determine the area of infiltration, type, and number of chambers. A target rainfall intensity, runoff rates, permeability, and hydraulic conductivity provided by the substrate on site needs to be determined prior to DIS implementation. Properly designed, the DIS can be a low-cost and low-tech solution for diminishing stormwater discharge and associated fecal bacteria loads to recreational beaches. The DIS can be installed entirely underground, reducing space requirements and promoting aesthetic attractiveness (Figure 23).

Figure 25. Photos of a dune infiltration system being installed at Kure Beach, NC (Source: Town of Kure Beach, NC).

4.4.2 Bioinfiltration / Bioretention

Bioinfiltration/bioretention is one of the infiltration system variations that provides runoff controls using surface storage, vegetation, planting soil, outlet controls, and other components to treat, detain, and retain stormwater runoff. These systems are designed to reduce stormwater runoff rate, volume, and pollutant loads as the stormwater runoff flows into the surface storage, ponds on the surface, and either gradually infiltrates into the native soil bed (i.e., bioinfiltration) or drains via underdrains (i.e., bioretention).

<u>Design Principles</u>

Standard bioinfiltration/bioretention systems (BIRS) are composed of several subcomponents including pretreatment, flow entrance, ponding area, plant material, organic layer (or mulch), planting soil medium, and positive overflow components. Figure 25 illustrates a typical BIRS design with underdrain and infiltration storage layer. BIRS are designed primarily to provide infiltration for relatively small volumes of stormwater runoff. The surface area is recommended to be approximately 3 to 6% of the CDA or less than 5 acres. If greater volumes of runoff need to be managed or stored, the system can be designed with an expanded subsurface infiltration bed made with stone or gravel underneath the planting soil medium. Pooling the water on the surface depression planted with trees, shrubs, and other herbaceous vegetation allows suspended solids and sediments to be settled and filtered before reaching the underlain planting soil layer. BIRS areas should be designed to completely drain within 72 hours after the end of a rainfall event and should not exceed a maximum ponding depth of 18 inches to avoid nuisance ponding conditions (e.g., mosquito breeding, odor). The planting soil is an engineered soil medium comprised of sand, soil, and organic matter. The organic nutrients spur vegetation growth and the additional storage within the mixed soil medium promotes infiltration and filtration of pollutants from the runoff flows. Plants also take up pollutants and plant superstructures promote evapotranspiration of the runoff water. The root system can also enhance infiltration of the surface runoff. The planting beds, including the surface mulch and mixed soil medium, should be between 18 - 36 inches deep but need to be designed to maintain at least 0.5 feet distance from the bottom of the BIRS to the top of the seasonal high water table. Underdrains are required if the measured permeability of the underlying soils is less than 0.3 in/hr.

<u>Benefits & Suitability</u>

BIRS come in a variety of configurations from relatively large and open vegetated basins to small-scale systems contained within flow-through planter boxes. The system is often installed for lot-by-lot stormwater management (i.e., “rain gardens”, Prince George’s County, MD) but the layout of BIRS facilities can be very flexible and increased size and an engineered overflow structure can be installed to manage larger sites. BIRS can be applied to most soils or topographies but designers must verify soil permeability to determine if the use of an impermeable liner or underdrain is necessary. Bioretention facilities can be integrated into already developed lots and sites such as parking lot islands, in landscaped areas around buildings, the perimeter of parking lots, and in other open spaces (Figure 26). To prevent damage to building foundations, risk of seepage, and contamination of groundwater aquifers, BIRS areas should keep distances of at least 10 feet from building foundations and property lines, 50 feet from septic systems, and 150 feet from private water supply wells. In addition to the stormwater management benefits of reduced runoff rates, volumes, and pollutant loads, BIRS can also promote wildlife habitat, urban heat island mitigation, and improved air quality. The selection of (native) plant species can provide for a wide variety of landscape designs mimicking natural ecosystems that are aesthetically pleasing.

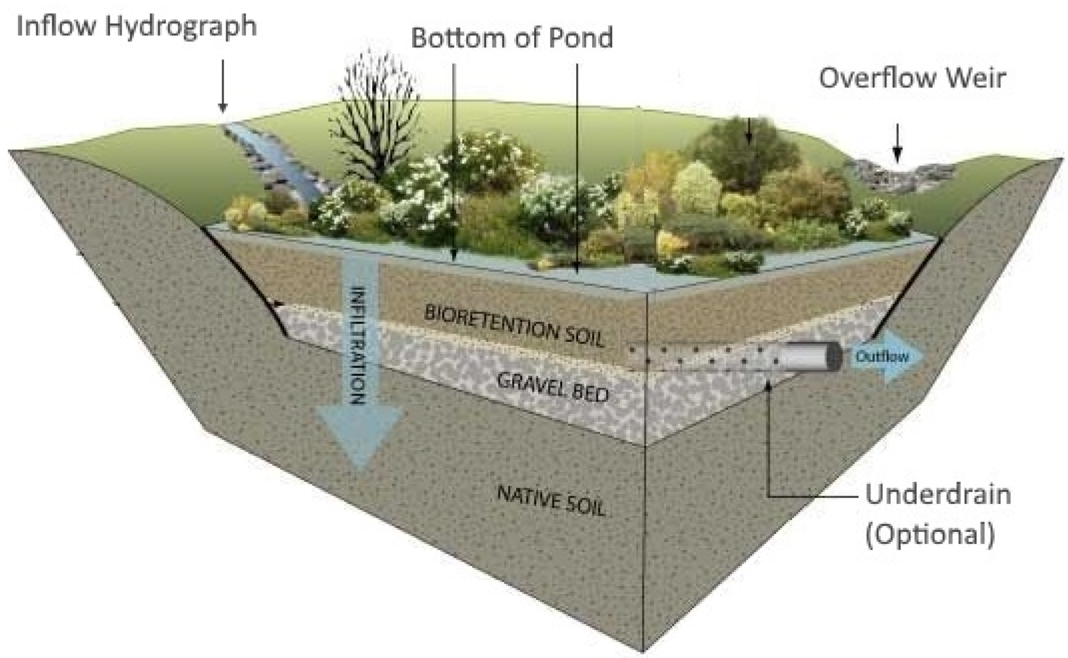

Figure 26. Rendering of bioretention design with underdrain and infiltration storage layer. (Source: www.hydrologystudio.com).

Figure 27. Photos of examples of bioretention in a parking lot (top left; source: Casey Patterson, Central Coast Low Impact Development Initiative), with drop curbs in a street setting (top right; source: National Association of City Transportation Officials), and as flow-through stormwater planter in a residential setting (bottom; source: Philadelphia Water Department).

4.5 Bio-Filtration (Open Channel Conveyance) Systems

Bio-filtration systems, or biofilters, are source control elements in stormwater management. They are designed to convey and treat stormwater flows in vegetated systems and to provide some degree of treatment, storage and infiltration prior to discharge to the storm sewer system. Bio-filtration systems are open channel and vegetate buffer systems designed to slow runoff, promote infiltration, and filter pollutants and sediments in the process of conveying runoff generated from a particular drainage area.

4.5.1 Vegetated Swales

Vegetated swales are broad, shallow channels in trapezoidal or parabolic shapes, densely planted with grasses and other herbaceous plants, shrubs, and/or trees. Vegetated swales are designed to attenuate and in some cases infiltrate runoff volume from adjacent impervious surfaces, allowing some pollutants to settle out in the process.

Design Principles

A Vegetated swale typically consists of a band of dense vegetation, underlain by at least 24 inches of permeable soil. Swales constructed with an underlying 12 to 24-inch aggregate layer (e.g., swales with infiltration trench) can provide significant volume reduction and reduce the stormwater conveyance rate. Two primary considerations for designing vegetated swales are channel capacity and minimization of erosion. Vegetated swales should be designed to convey flow volumes associated with the 2- and 10-year storm events at non-erosive velocities (generally less than 6 fps) for the soil and vegetative cover provided (Ellis et al., 2014). Velocity and flow depth exceeding the channel capacity may impair the effectiveness in treating runoff or preventing erosion in the channel. Vegetated swales are sized to temporarily store and infiltrate the 1-inch storm event, while providing conveyance for up to the 10-year event with some freeboard (6 inches recommended). The maximum CDA to most swales should be 2.5 acres, and preferably less. Longitudinal slopes of the site for vegetated swale application should be less than 4% but channels designed with longitudinal slopes of less than 1% should be monitored carefully during construction in order to avoid flat areas that cause standing water issues. The hydraulic head, the elevation difference between the inflow point and the outflow point or storm drain invert, should be between 3 and 5 feet. Check-dams are recommended for vegetated swales with longitudinal slopes greater than 3%. Check-dams are a series of small, temporary dams constructed across a swale, drainage ditch, or waterway to counteract erosion by reducing water flow velocity. Check-dams constructed with regularly-spaced natural wood, concrete, stone, or earth at a height of 6 to 12 inches can be employed in order to reduce the effective slope of the channel and lengthen the contact time to enhance filtering and/or infiltration.

Vegetated swales come in a variety of shapes but are typically categorized into grass channels, dry swales, or wet swales (Figure 27 and Figure 28). Grass channels are linear vegetated ditches used to treat and reduce flow velocities of stormwater runoff. Planting grasses is relatively inexpensive and the linear nature makes the grass swales applicable to nearly everywhere to provide a modest amount of runoff filtering and flow attenuation within the stormwater conveyance system. Grass channels are typically most suitable for sites with relatively mild side and longitudinal slopes and can only provide minor runoff volume reduction due to the lack of storage volume. Grass channels do not have an engineered filter media and hence provide only a relatively low level of pollutant removal. Dry swales are essentially linear bioretention cells covered with turf and other surface vegetation (tall meadows, herbaceous plants, or trees). Dry swales utilize pre-mixed soil media filter below the channel (as was the case for bioretention) that can temporarily store and filter the desired design storm volume. While some designs allow for the runoff to infiltrate into underlying soils, in most cases, the runoff treated by the soil media flows into an underdrain systems. Dry swales in soils with infiltration rates of less than 0.3 inches per hour will need an underdrain. The bottom of dry swales and grass channels needs to be at least 0.5 feet above the seasonally high groundwater table in order to prevent groundwater contamination or practice failure. Wet swales are linear wetland cells that incorporate shallow, permanent pools or marshy conditions that can sustain wetland vegetation. They can be designed to intersect shallow groundwater as long as the water table does not inundate pools or reduce available runoff storage, as to maintain wetland plant communities. Saturated soil (hydrologic soil group C or D) and wetland vegetation within wet swales provide an ideal environment for gravitational settling, biological uptake, and microbial activity.

Figure 28. Photos of vegetated swale design options. Grass channel, dry swale, and wet swale (from left to right) alongside roadways and parking lots (Source: Ellis et al., 2014).

Figure 29. Photos of dry swales in urban settings. A shallow vegetated swale with check-dams alongside a distributor (pervious) road (left, source: Cambridge City Council) and a typical application in a residential neighborhood in Chambers County, Texas. (Source: Barrett et al., 2014).

Benefits & Suitability

Vegetated swales can reduce peak flow at the discharge point by increasing travel time and friction along the watercourse and provide some infiltration and water quality benefits. Vegetated swales are used as an environmentally superior alternative to conventional curb and gutter conveyance systems. A swale can be more aesthetically pleasing than a concrete or rock-lined drainage system and is generally less expensive to construct. Swales are typically used for runoff conveyance or pretreatment to other stormwater management systems as they can be designed to fit into many types of landscapes including relatively narrow corridors between utilities, roads, parking areas, etc. They may be applicable in many urban settings, including parking lots, commercial and light industrial facilities, and residential settings. They are also ideally suited for the coastal flat topography where stormwater is conveyed primarily in open channels. Generally, vegetated swales require large footprints and therefore it may be impractical to implement them in densely developed areas. Typical vegetated swales cannot be used on sites with steep slopes.

4.5.2 Vegetated Filter (Vegetated Buffer) Strips

Vegetated filter strips (VFS), or vegetated buffer strips, are vegetated sections of land similar to grassed swales, except they are essentially flat with low slopes, and are designed only to accept runoff as overland sheet flow directly from adjacent impervious surfaces. As water flows onto and across the VFS, usually towards a swale or filter drain, the VFS functions to slow down runoff velocities, filter out sediment and other pollutants, and provide some infiltration into underlying soils. Filter strips were originally used as an agricultural treatment practice, and have more recently evolved into an urban practice. Filter strips are generally a sensible and cost-effective stormwater pretreatment option applicable to a variety of development sites, including roads and highways.

Design Principles

A VFS is a gently sloping area of vegetation that is planted intentionally with turf grasses, shrubs, or other indigenous woods and trees to help remove sediment and other pollutants from runoff water. The general design goal is to produce uniform, shallow overland sheet flow across the entire filter strip. Figure 29 presents a typical layout and example of VFS applied in residential and commercial areas. Sheet flow is the flow over plane surfaces that occurs in the headwater of streams. After a maximum of 300 feet (100 feet in urban areas), sheet flow usually becomes shallow concentrated flow (The U. S. Department of Agriculture, 1986). Concentrated flows can be distributed along the width of the strip using a gravel trench or other level spreader devices (curb stops, earthen berms, etc.) to promote sheet flow conditions (Aransas County, 2011). The maximum CDA must be less than 5 acres and a ratio of CDA to filter strip area must not exceed 6:1. VFS effectiveness can be enhanced by installing berms and retentive grading at the toe of the slope, perpendicular to the flow path. The minimum dimension of the filter strip (in the direction of flow) is a function of the slope, vegetative cover, and soil type but no less than 25 feet is recommended. VFS slopes should not exceed 10% but slopes less than 5% are recommended. Minimum VFS width (in the direction of flow) should equal the width of the CDA and a maximum contributing drainage area slope is generally less than 5%. The maximum width of the contributing impervious area should not exceed 72 feet. VFS should extend along the entire length of the contributing area and the slope (in the direction of flow) of the top of the filter strip must be very small (less than 1%) and gradually increase to the designed value to protect from erosion and undermining of the device. The entire extent of the VFS should lie above the elevation of the 2-yr, 3-hr storm of any adjacent drainage area. The seasonal high watertable should be at least 2 to 4 ft lower than any point along the filter strip (Barrett et al., 2014). To avoid flow channelization and maintain performance, a VFS should contain dense vegetation with a mix of erosion resistant, soil binding, and deep root penetration species. Diverse native vegetation of varying physical types is preferred.

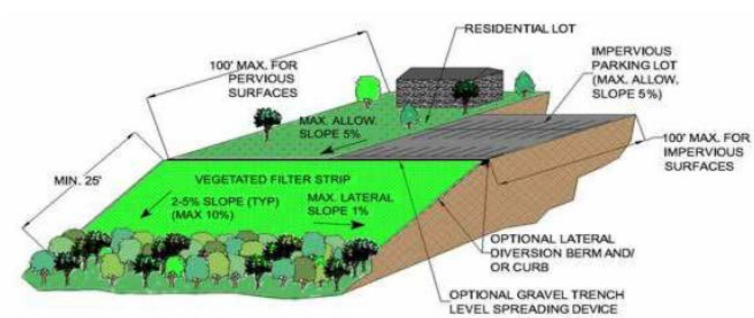

Figure 30. Schematic and photo of vegetated filter strips (VFS). A conceptual illustration of a VFS system (left, source: Pennsylvania Stormwater Best Management Practices Manual, 2006), and an urban VFS providing a buffer between an impervious roadway and a vegetated swale (right, source: Chesapeake Stormwater Network).

Benefits & Suitability

VFS are intended to treat stormwater sheet flow from adjacent pervious and impervious areas by reducing runoff velocity, trapping sediment and pollutants and, in some cases, infiltrating a portion of the runoff into the ground. VFS are frequently used as a “pretreatment” system prior to discharge to a variety of BMP features, including natural buffer areas, vegetated swales, and infiltration basins. They can be used along toes and tops of slopes and at outlets of other stormwater management structures. Vegetated filter strips can also provide aesthetic benefits, stormwater storage, and wildlife habitat. In addition to stormwater management, VFS can add recreational value with opportunities to incorporate trails into their design. Filter strips are best utilized to treat runoff from roads and highways, roof downspouts, small parking lots, and pervious surfaces. It is critical that plant materials are appropriate for soil, hydrologic, light, and other site conditions. No runoff should be allowed to flow across the filter strip until the vegetation is established (at least three months after seeding). Concentrated flow and soil compaction can damage the filter strip and compromise the strip effectiveness. Therefore, the filter strips may not be used in high-use, ultra-urban areas unless adequate precaution devices such as level spreaders are provided in order to prevent the development of the concentrated flows, and hence, to prevent erosive flow conditions. Other precaution structures includes the signage, fences, and placement of sidewalks that can be used to minimize disturbance of the filter strip.

4.6 Passive Beach Dewatering Systems (Beach Drainage Improvement)

Heavy rainfall events and frequent overland runoff discharge to sandy beaches can cause significant flooding, sediment removal, as well as beach and dune erosion. Such adverse runoff impacts on coastal sandy beaches can be mitigated by improving the beach drainage capability. When the drainage capacity through a permeable layer is impaired either by a relatively long period of saturation by frequent wave run-up and stormwater overflow or due to a mild beach slope, it promotes seaward seepage through the upper beach face and sand fluidization. Furthermore, the wet sand of a drainage-impaired beach tends to be more to rapid erosion once disturbed by wave action. Furthermore, it lacks the capacity to infiltrate water during wave uprush which leads to a higher erosion potential for the backwash (Katoh and Yanagishima, 1996). The beach erosion associated with wave and overland runoff can be mitigated by enhancing hydraulic conductivity in the drainage watercourse. A passive drainage system provides a permeable layer under the beach through which the surface water can be infiltrated into the sublayer by the effect of percolation and then conveyed offshore as an internal flow driven by gravity. A passive drainage system, in contrast to an active system that utilizes powered pumps to transport beach water to a tube, is a relatively cost-effective measure to drain groundwater seaward purely by gravity through a permeable layer below the beach surface.

4.6.1 Gravity Drainage System (Strip Drainage System)

Design Principles

Figure 30 shows the prototype Gravity Drainage System (GDS) installed on Dee Why Beach, New South Wales, Australia in March 1991 (Davis, et al., 1992). The GDS incorporates an array of shore-normal "strip" drains buried horizontally 2 – 3 meters underneath the surface sediment layer and ends in the swash zone area of the beach. The elevation of the landside inlet and mean water level at the seaward outfall need to be determined to ensure down-gradient groundwater flows that discharge naturally offshore by gravity through the artificially built permeable drain layer. The drain strip frames are built with corrosion-proof expanded metals nesting in a crossing pattern and stacked vertically with some empty space (e.g., 20 % of the longshore span of each strip). The strip frame is enclosed by a geotextile fabric for additional filtering capability and protection from large debris and other impacts.

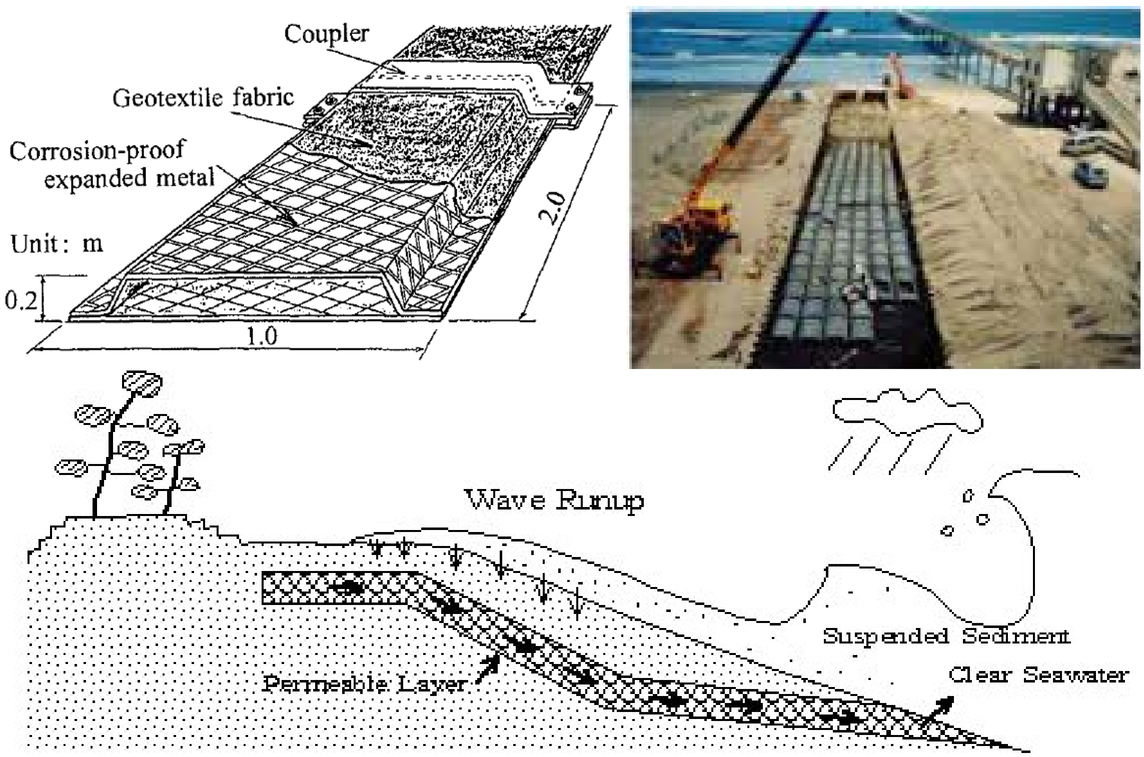

Figure 31. Schematics and photo of beach gravity drainage systems (GDS). Shown are a general layout (top-left) and installation (top-right) of the subsurface permeable layer, and conceptual illustration of dewatering mechanisms of a GDS (bottom, Source: Port and Airport Research Institute).

Benefits & Suitability

GDS can be installed in the upper beach face between areas impacted by swash and inner surf zone flows where frequent ponding leads to sediment erosion and washoff. GDS intend to enhance the soil infiltration capacity and move the point of water discharge offshore to the shoreface at the end of the drain strip. Expected effects are the enhanced absorption of surface flow associated with wave runup and overland runoff and reduction of wave loads during rundown. GDS is intended to promote an accreting beach environment as more sediments carried onshore by the incident waves can remain on the upper beach and beach erosion due to surface water runoff or wave rundown can be mitigated with enhanced sublayer drainage capability. Such benefits are reportedly greatest on micro-tidal (< 2 m range) beaches subject to low and moderate wave energy (i.e., significant wave height H_s< 4.5 meters or 14.8 feet, www.nodc.noaa.gov/GTSPP, accessed on 08/01/2018) where exposure of the buried drain strip during large-wave draw down is not expected (Scottish Natural Heritage, 2000). GDS may be installed in connection with upstream rainwater runoff treatment/collection systems. The overland discharge to the beach can be received by these systems and partially conveyed through the permeable layer within the GDS before causing severe beach erosion and scarping. Because GDS is buried below the beach surface, no visible impedance is created on the beach plain. Anecdotal evidence suggests that damage to the seaward ends of individual drains during storms can reduce the efficiency of the system but GDS still continue to aid expedited recovery of an eroded beach (Greg A Davis et al., 1992; Katoh and Yanagishima, 1996). Installation costs, while dependent on the number of strips to be laid on each site, can be relatively low but maintenance and management commitments are relatively high. General design criteria have not been established, yet, and hence, the design parameters (dimensions and number of drain strips, depth of burial, etc.) need to be determined as a function of the prevailing nearshore hydrodynamics (tides and waves) and sediment dynamics (permeability, erodibility, slope) of the target site.

4.6.2 Beach Drainage Systems (Toepassing Drainage Systems)

Design Principles

Figure 31 provides the general layout and conceptual illustration of the principles behind Beach Drainage Systems (BDS). In essence, BDS are perforated “toepassing” drain pipes buried in parallel to a coastline, below the upper beach surface within the high tide swash zone. The drainage pipes are installed with a small longitudinal slope in order to convey water to a collection point by gravity. From there, it is pumped out and then either discharged to the sea or redirected inland for reuse, for example, in aquariums, saltwater lagoons, etc. (Dienst Weg- en Waterbouwkunde, 1994). A geotextile sleeve is laid into excavated trenches and encloses the drain pipe to filter sand from the seawater collected in the drains. Pumping facilities must be appropriately housed and discharge of collected water should be designed to minimize any interference with natural beach processes. In principle, BDS aims to promote a favorable condition for sediment deposition by lowering the ground water surface level and, therefore, enhancing beach permeability and reducing rundown flow velocities by increasing the portion of wave uprush or surface runoff discharged as seepage flows. It also aids beach stabilization by effectively hardening the sand on an unsaturated zone in the upper beach by moving the point of seepage outflow further away from the toe of the beach. The relatively pure seawater filtered by surrounding sand and additional gravel filters can be discharged back to the sea or can be redirected inland for re-use in various purposes (e.g., feed for heat pumps, land-based aquaculture, wetland oxygenation, etc.).

Benefits & Suitability

Such systems are reportedly most effective in areas with a low tidal range (less than 2 m) and low to moderate wave conditions (Hs less than 0.6 m) (Ciavola et al., 2009). The method is quite simple to implement and requires largely invisible, buried structures. The most visible components of such systems are the collector wells and pumping stations. The major cost will occur for installation and maintenance of the pumping facilities while a pump with a floater switch can improve the cost efficiency by operating sensitively to the water level inside the collector well. A single system provides most effective benefits when the pipework is laid at lower tide levels over relatively short lengths of shoreline (100 m to 400 m). Small embayment or a discrete length of a beach separated by headlands was found to be appropriate sites (Scottish Natural Heritage, 2000).

The successful application of BDS is expected to enhance sand deposition and increase upper beach volume, however, systems installed based on poor site selection, inadequate design, and lack of management could result in little to no benefits to beach stabilization. Currently, there are no adequate long-term monitoring results undertaken in the field at a frequency sufficient to understand the performance or life expectancy of the system as a function of location, depth, size, and number of parallel drains in response to morphodynamic beach conditions (Leonardo Damiani, 2011; Bain, et al., 2016). Therefore, professional hydraulic design is required to minimize operational costs and impairment during storm events, and to optimize the efficiency of beach dewatering and beach stabilization.

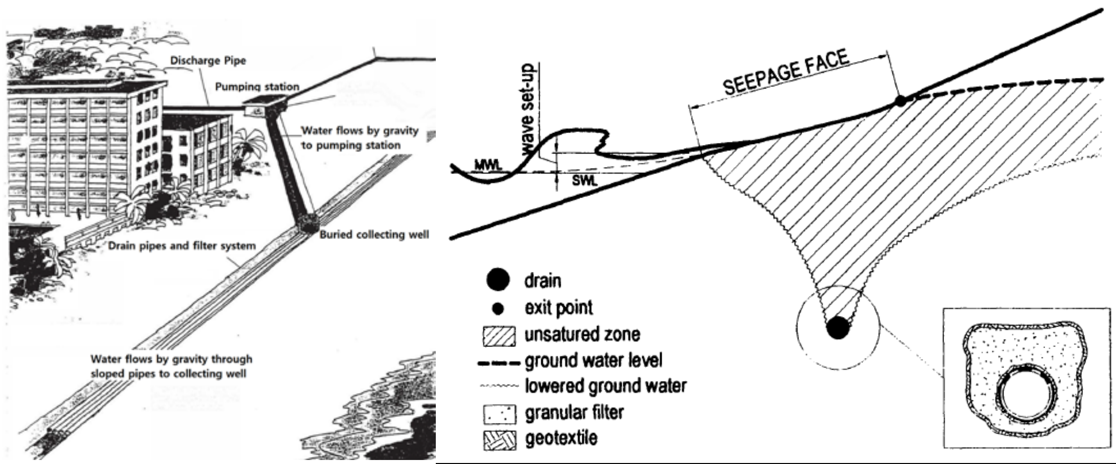

Figure 32. Schematic drawings of beach drainage systems (BDS). Shown are a general layout (left; source: Dienst Weg- en Waterbouwkunde, 1994) and conceptual cross-sectional illustration of the dewatering mechanisms (right; Ciavola et al., 2009).

4.6.3 Pressure Equalizing Modules (Vertical Drainage System)

Pressure Equalizing Modules (PEM) use porous vertical tube media to penetrate and connect subsurface soil layers with different levels of flow resistance to promote groundwater conveyance and to modulate groundwater pressures by reducing the watertable on the upper beach.

Design Principles

Pressure Equalizing Modules (PEM) use cross-shore arrays of vertical drainage pipes installed underneath the bed surface along cross-shore beach transects between the mean high water line (e.g. dune line) and mean low water line (Jakobsen and Brøgge, 2008). Figure 32 shows an example of PEM and an illustration of the underlying mechanism. PEM are used to provide a watercourse for surface water infiltration into the sublayers and for subsequent groundwater flows toward relatively lower flow resistance (typically a coarser layer). The local lowering of the groundwater table enhanced by this dewatering process, however, can results in a drier beach where sediments in the surface substrate layer can become more susceptible to suspension by winds. Aeolian processes can then carry fine sands up toward the upper beach near the dune line while leaving coarser sand on the lower beach which in turn promotes higher permeability and increased drainage capacity. A drain pipe is composed of permeable layers (slits) at various heights, allowing for water to enter but preventing sands from getting in. The air inflow through the porous filter cap at the top expels the water sitting inside the tube, forcing the dewaterization across the depth of the tube. The size of a single PEM tube varies depending on the site condition but typically has dimensions of 2 m in length and 0.06 m in diameter.

Benefits & Suitability

This patented PEM technology (i.e., “Ecobeach”,“ EcoShore®”) is a “soft” dewatering solution applying a small modulation to achieve a balance in the beach system, resulting in the accretion of sand. PEM are placed subsurface and are invisible except after an unusually heavy storm, where they may be visible during a short period of time. PEM are cheap to install and require no power to operate. PEM are reported to help establish a new equilibrium profile with a wider and higher beach within a year post storm (Christensen, 2016). The system was reported to function most effectively when PEM were remained within the sand drift zone where the available sediment that can be carried onshore by coastal waves and winds is sufficient. Therefore, it was recommended to install the PEM to coincide with (soon after) the beach nourishment. PEM have been implemented on various sandy beaches worldwide since 2007 including Hillsboro Beach, FL, USA and Egmond aan Zee, the Netherlands, with demonstrated effectiveness in beach stabilization and sediment accretion. However, the working mechanism is still in the hypothesis stage and therefore, a local-scale pilot study is recommended prior to full installation to obtain detailed field measurements of changes in ground water level, sublayer pressure, and spatial composition of sediment along with beach and dune profiles (Pieterse, 2009).

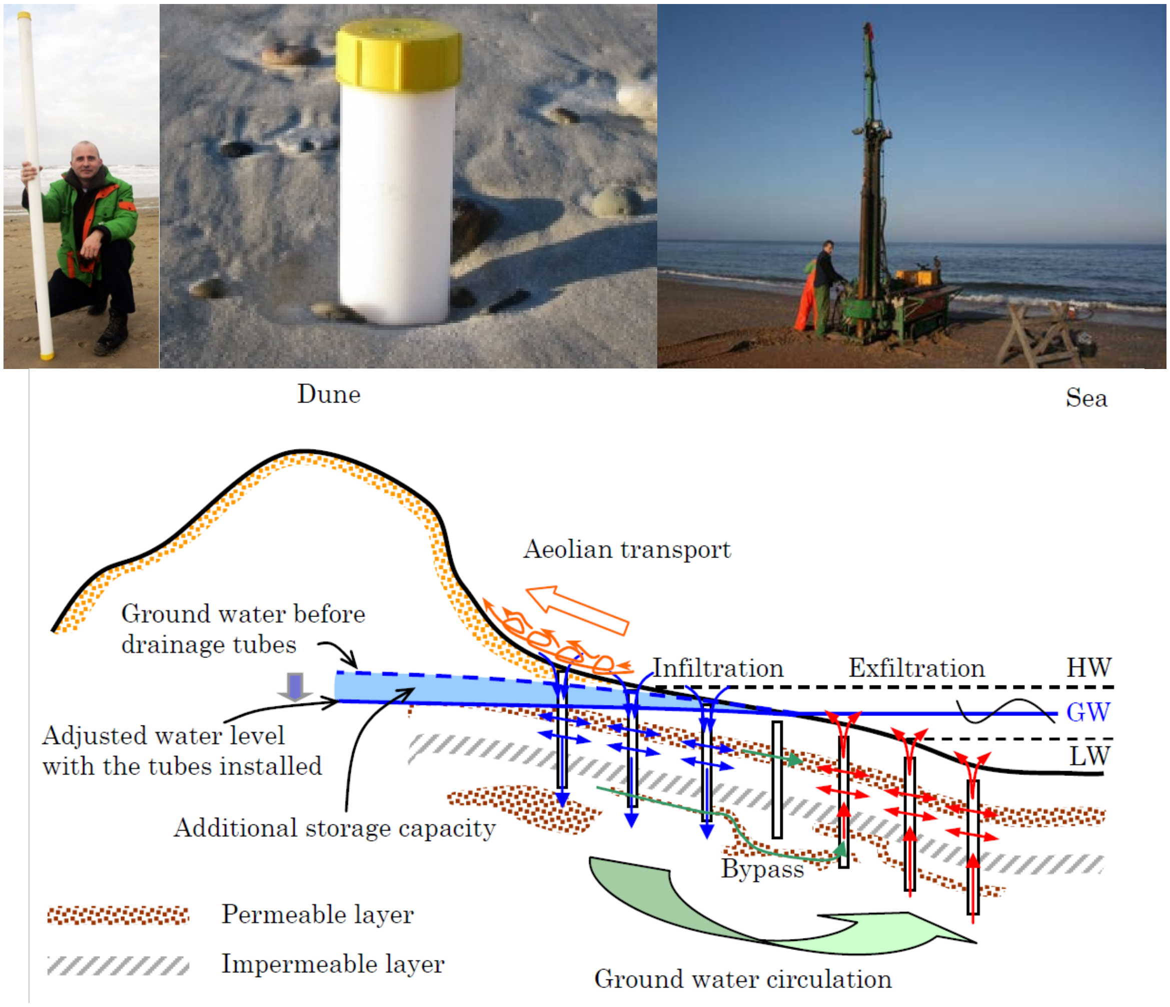

Figure 33. Photos and cross-shore schematic of PEM examples. Shown is a single module and installation of PEM on a sandy beach (top; source: Royal BAM Group), and a conceptual cross-shore illustration of the vertical drainage tube arrangement implemented through PEM (bottom; source: Ekkelenkamp, 2012).

4.7 Armoring and Diversion Systems (Outfall Protection and Slope Stabilization)

4.7.1 Rock and Timber Revetments

A revetment is typically a sloping, permeable structure constructed with natural stones or concrete blocks built to protect the base of a beach scarp, a foot of a cliff or a dune, a dike or a seawall against erosion by wave action, storm surge and currents. A revetment is often a supplement to other types of protection such as seawalls and dikes for scour problems associated with impermeable surfaces of the concrete structure. Scour impact can be reduced by placing a porous armoring medium on the seafront base. Revetments can provide the effect of coastline stabilization and dune protection by enhancing wave energy absorption and minimizing reflection and wave run-up. Revetments can consist of different kinds of materials including rocks, timbers, and some other interlocking, permeable concrete slabs or rock baskets called gabions.

Design Principles

Revetments can be built with different kinds of materials and those utilizing gabion baskets, armor rocks, and timbers are the most typical schemes that can be installed in sandy dunes and beaches via relatively low impact development. Unit material size, face slopes, and crest shape (elevation and width) must be determined depending on site wave and landscape conditions, cross-sectional design under consideration, acceptance of risk, and the availability/cost of armoring materials. Figure 33 shows examples of different types of revetment application installed for coastal dune protection.

Rock revetments take on the form of either roughly placed riprap slopes or extended engineered structures that are designed to provide protection for short sections of severely eroded dunes or long-term erosion mitigation for long lengths of shoreline. A rock revetment typically consists of a rock structure constructed within a shallow trench. The base layer is wrapped by a geotextile to prevent the migration of sand upwards and the settlement of the rocks into the beach. The toe of the engineered rock structure should be set below the lowest expected beach level (i.e., a high water mark of ordinary spring tides) in order to avoid localized scour. The structure crest elevation must be above the wave run-up limit but some inevitable potential overtopping impacts during extreme events should be taken into consideration for the cross sectional design. In general, widely graded rocks from small boulders up to armor rocks are placed at least in double layers to form a sloping face that resembles natural dune slopes (within a 1:1.5 to 1:3 range). To increase hydraulic efficiency, the rocks should be placed randomly to form a rough surface with large voids. Safe access routes should be built for public safety and accessibility to beaches. Smaller-scale riprap slopes that can be constructed along estuary shores or well protected coastal sites are subject to regular storm damage. Therefore, regular maintenance planned to avoid harmful damage to dunes and beaches by heavy equipment use is necessary. Other measures such as burial of the riprap slope shall be implemented to encourage structural stabilization and fencing, thatching, and planting of salt-tolerant vegetation on the upper slope to reduce runoff velocities from the overland and to promote native vegetation growth.

Timber revetments utilize treated softwood or hardwood and serve as a temporary permeable upper beach wave barrier or impermeable breastwork for dune erosion protection. The flexibility of timber as a construction material allows the timber revetment to apply for small to large schemes although, due to limited sustainability and increased costs of hardwood materials, recent use of timbers is limited to small schemes in relatively low-energy areas. Timber breastwork consists of impermeable structures that run in parallel to the shoreline and should be built straight (vertical) above the limit of normal wave run up. Timber wave barriers can be built lower on the foreshore, so far as to be above the normal high spring water line, and can be vertical or sloping structures. The permeable barriers should maintain a ratio of opening to the solid blockage between 0.2 and 0.5. The face slope should be designed so structures can remain stable under wave impacts (Scottish Natural Heritage, 2000a).

Figure 34 Photos of revetment variations for coastal dune protection. Shown are gabion mattresses and gabion walls (left), a rock armor revetment (center), and timber breastwork (right) in coastal environments (source: Scottish Natural Heritage, 2000).

Benefits & Suitability

In general, revetments are a passive coastal protection measure that can be used at locations exposed to ongoing erosion or as a supplement to seawalls or dikes at locations exposed to both erosion and flooding where it is not cost-effective or environmentally acceptable to provide full protection using seawalls. To minimize the erosion to the fronting beach and adjacent areas by wave reflection or focused runoff, the revetment should be located as far landward as possible. Feathered endings turning the revetment face back into the dunes and burying the end into the dune face can help minimize local sour and possible outflanking problems. However, a revetment can provide only minimal protection against flooding.

Rock revetments provide robust, long-term protection for important backshore assets. However, compared to the revetments built of gabions or timbers, they are usually more expensive to construct but construction costs for rock revetments are heavily influenced by the availability of suitable material and transport methods. Rock structures can be assumed to have an unlimited life with respect to economic assessments with moderate effort of maintenance, while smaller rip-rap slopes will require regular maintenance costs to be included in the budget. Rock structures can significantly alter dune systems permanently by providing focus for future marine erosion and preventing the sand from building up over the rocks.

Within estuaries or on low energy beaches, timber revetments with softwood structures may provide protection with a life expectancy of 5 – 10 years while hardwood timber may last 25 – 30 years. Due to the construction flexibility, timber revetments can offer a variety of design options and can be incorporated into recreation management schemes that are often readily accepted by the public. While depending on design, dimensions, and quality of materials, costs for construction and maintenance is cheaper than building seawalls or rock revetments. Where permeable wave barrier revetments are built on the active upper beach, exposure is possible. Damage to the structures should be repaired rapidly to maintain the effectiveness of the scheme.

4.7.2 Gabion Revetments

Design Principles

Gabions are wire mesh baskets or mattresses filled with cobbles, crushed rock or other locally available materials. Gabion revetments are comprised of an armor layer, a filter layer, a toe stone and the crest. A suitable geotextile should be placed underneath the gabion fills to prevent sand washout. Gabions made up with non-angular (i.e., less mobile) stones need to be carefully packed by PVC-coated wire or galvanized wire of a larger diameter in active energy conditions with wave heights up to 2.0 m or 6.5 ft (USACE, 1986) in order to prevent damage from abrasion or corrosion. Gabions can be placed as sloping mattresses (i.e., Reno Mats) or as near vertical walls (cubic baskets). Figure 34 shows the general layout of the gabion mattresses and gabion boxes installed for protection and reinforcement of the coastal dune. Sloping gabions are preferred for dune protection purposes, as they will tend to trap wind-blown sand and allow the growth of vegetation under favorable conditions. Near vertical gabion walls are more likely to suffer toe scour and structural collapse are much more obtrusive to the dune landscape and unlikely to attract sand to promote dune growth or build-up of a new foredune. The slope needs to be determined so it conforms with (surrounding) natural dune slopes while minimizing the construction footprint so the effectiveness in wave energy absorption is optimized. A slope of 2:1 (H:V) has been found to be a reasonable compromise (Massachusetts Office of Coastal Zone Management, 2014; Scottish Natural Heritage, 2000b).

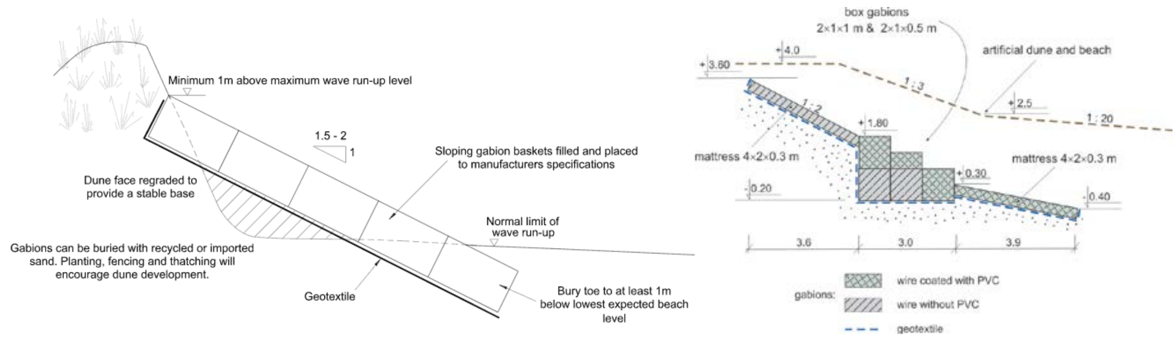

Figure 35. Schematics of typical cross-sections of gabion revetments. Shown are gabion mattresses laid over a regraded coastal dune face (left; source: Scottish Natural Heritage, 2000) and gabion boxes built into an artificial dune for reinforcement (right; source: Rafal Ostrowski from coastalwiki.org, Ostrowski, 2008).

<u>Benefits & Suitability</u>

Gabion revetments are intended to provide short term (5 – 10 years) protection from backshore erosion and hence their application should be restricted to the upper part of sandy beaches, above the run-up limit of normal waves. Gabions placed as near vertical cubic baskets are intended for bank or cliff stabilization but more vulnerable to toe scour and structural collapse in addition to being more obtrusive to the beach and dune landscape. Gabion revetments may not be appropriate for sites with more than 1 meter annual erosion rate. Regular basket maintenance is required to maximize the life of gabions. Damaged or improperly maintained gabions can become dangerous to the general dune/beach environment and to adjacent baskets. Buried gabion box revetments built into an artificial dune may naturally blend into the dune/beach landscape while allowing a natural dune/beach system to develop under less extreme conditions. Burial in recycled sand, combined with vegetation transplanting, thatching and/or fencing can enhance the recovery of the dunes over the gabions. As they utilize locally available materials, a gabion revetment has a relatively low capital cost and can be a short-term alternative to rock amour structures in areas where large rocks are not available at an acceptable cost, or where long-term protection is not appropriate. Safe public access routes should be provided across the gabions.

4.7.3 Bio-Thatching (Rough Ramp Systems)

"Armoring" or "hard" coastal engineering structures such as seawalls and revetments can provide storm damage protection and erosion control from waves, tides, currents, and storm surge. However, vertical or sloping elements constructed of concrete, stone, or composed of rocks (called "rip rap") of such solid shoreline protection systems can significantly alter the coastal system and exacerbate sediment erosion by reflecting waves or creating standing wave patterns. Overland runoff or wave overtopping pressure building up behind the wall or upper bank can be potential cause for structural failure. Low-impact, erosion mitigation measures can be incorporated into the design of the hard revetment structures. This can include the introduction of rough surfaces or redirecting of overland runoff or overtopping flows. Bio-thatching is a coastal bioengineering practice that utilizes natural, biodegradable erosion-control products, such as bio-mats or coir rolls in combination with deep-rooted plants. It can be used to prevent erosion from overland runoff and wave action on the beach side toe of hard coastal defense structures.

Design Principles

Bio-thatching utilizes natural fiber blankets (i.e., biomats) or cylindrical rolls (i.e., coir rolls) made of natural, biodegradable materials, such as straw, burlap, and coconut husk that are held together by loosely woven mesh or coir twine. Figure 35 illustrates the installation of the bio-thatching systems made of a natural fiber blanket (left) and coir fiber rolls (right) for slope protection. Figure 36 shows examples of the bio-thatching system installed for the coastal dune stabilization projects. Bio-thatching provides direct, physical protection to reduce erosion of bare soils from wind, waves, and overland runoff and promotes growth and settlement of native, deep-rooted vegetation and hence stabilization of eroding shorelines. Biomats and coir rolls can be installed on almost any non-vegetated coastal bank or bluff but are most effective in areas with higher beach elevations with some dry beach at high tide, where the toe of the bank is not constantly subject to erosion from tides and waves. Rolls of natural fiber blankets are recommended to be placed from the top to the bottom of the slope with overlaps of 6 to 12 inches to prevent exposure of the ground surface. Coir rolls are typically installed at the toe of the bank and at the base of or next to hard structures (i.e., seawalls and revetments) to serve as a physical barrier to waves, tides, and runoff flows and help reduce erosion of exposed sediments. Slope stabilization is essential for success of bio-thatching projects. The slope stabilization needs to be achieved before the biomats or coir rolls are installed by either adding fill at the bank toe or re-grading the top of the bank so the base slope becomes less steep than the upper portion of the bank. A salt-tolerant seed mix is spread across the area before the natural fiber blanket or coir roll is secured and then live vegetation with extensive root systems is planted directly into the bio-thatching products and surrounding area. The dense root systems help reduce soil settlement and limit erosion from rain, wind, tides, and waves. They also reduce the rate and quantity of upland water runoff by taking up water directly from the ground and breaking the impact of raindrops. Pedestrian access walkways should be installed to prevent trampling of plants, especially before the plants are established. The elevation and dimensions of the access structure should be determined as to minimize shading impacts on vegetation. If surface runoff is causing erosion, natural fiber blankets are typically installed over the entire surface of the non-vegetated slope from the top to the bottom while upland runoff flow should be reduced and/or redirected to give newly planted vegetation the best chance of survival. The number of rows and the individual diameter of coir rolls needed should be determined based on the site condition. Generally, it varies from one or two rows of 12-inch-diameter coir rolls for sheltered, relatively mild sloping sites to multiple rows of 20-inch-diameter rolls in more exposed areas and on steeper banks.

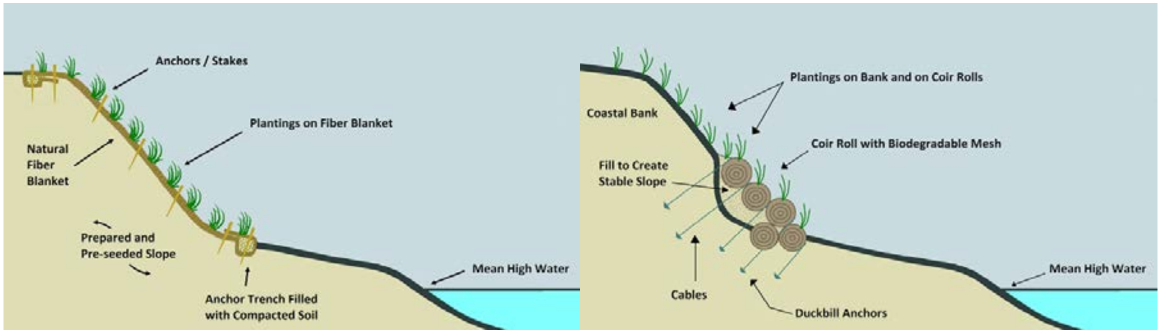

Figure 36. Schematic cross-sections of slope protection by bio-thatching systems. Shown are illustrations of a natural fiber blanket (left) and coir fiber rolls (right) installed on a coastal bank (source: Massachusetts Office of Coastal Zone Management).


Figure 37. Photos of bio-thatching (left) and fiber coir roll (right) applications for coastal dune stabilization projects (source: Massachusetts Office of Coastal Zone Management).

Benefits & Suitability

Bio-thatching products made with natural materials that are locally available can be a low cost, effective measure for erosion control on sloping faces of dunes, coastal bluffs and banks, or the seafront base of hard coastal engineering structures. Natural fiber blankets and coir rolls absorb much more wave energy than other hard shoreline stabilization structures while allowing some natural erosion from the site as it would be essential for supplying the sediment to down-drift areas. They can be installed without use of heavy machinery or skilled labor while preserving the natural character and habitat value of the coastal environment. Natural fibers used to make the biomats and coir rolls will disintegrate over time (e.g., typically over 6 to 24 months for biomats and 5 – 7 years for coir rolls). However, synthetic materials or wire meshes used for netting of high-density coir rolls do not easily degrade and can cause significant adverse impacts to the coastal environment (e.g., entangle wildlife, disrupted navigation, and harm to recreational beach users). Bio-thatching will not prevent erosion on unstable slopes or in areas subject to erosion from high tides or storm waves. Bio-thatching should not be undertaken on steep, freshly-eroded slopes and a maximum slope of 1:2 is recommended. Thatching will be quickly damaged by wave action and should not extend seaward of the line of normal wave run-up. Invasive plants that thrive at the expense of native species should be removed and replaced with appropriate native plants. Bio-thatching is most effective when biomats or coir rolls are placed in close contact with the soil or sediments. Bio-thatching will require ongoing maintenance and repair to ensure establishment of vegetation in the initial phase, to ensure contact with the substrate in areas of erosion, and to ensure the stabilized status of the slope or soil fills. Natural fiber blankets and coir rolls are frequently used together with other stormwater runoff erosion management techniques.

5. SUMMARY AND DISCUSSION

This chapter provides a summary of all reviewed BMP systems in table form (Table 11) and offers some supporting discussion. It includes a qualitative assessment of their capabilities in offering runoff peak control, reducing runoff quantity, and handling runoff pollutants as well as a rough qualitative assessment of relative construction and maintenance costs. Furthermore, the feasibility for implementation at the three Galveston Island project sites investigated in this report is indicated.

5.1 Qualitative Evaluation of Design Alternatives for Galveston Island Beaches

The following categories are used to assess the suitability of a certain mitigation design for a specific Galveston beach site: Effectiveness in controlling peak rate, quantity, and quality of the runoff discharge flow and aesthetic fit into the local setting. Three rating categories were used (high, moderate, and low). Relative cost ratings for construction and maintenance are also provided but cost was not incorporated into the suitability assessment of a certain BMP system for a specific Galveston Island beach site (Table 11).

The "runoff erosion control" category assesses the effectiveness of a given BMP design in mitigating beach and dune erosion and the formation of scour channels from the runoff. A design effectively delaying the impact of runoff discharge downstream of the source will be rated high. High ratings are also given to designs that effectively reduce runoff discharge velocities or erosion potential downstream by providing additional roughness or additional armoring effects.

"Runoff quantity control" rates the capability of a specific mitigation design to reduce surface water volume before runoff flows can reach the downstream pour points on the beach. Design options that promote evapotranspiration or transformation of surface flows into groundwater flows receive higher scores here. "Runoff quality control" relates to the ability of a mitigation design to effectively reduce the level of suspended solids and pollutants in the runoff flows by means of filtration and temporary or permanent storage. Practices that provide mechanisms to divert or dilute the concentration of the runoff discharge or to remove pollutants through filtration, adsorption, and biological conversion are also rated high in the quality control category.

The "construction cost" category considers both the installation cost and the potential footprint of the design. For example, if a design option requires a large footprint to be implemented, it was considered to have "high" construction cost even if the installation cost of the system was found to be moderate (e.g., infiltration basin). The "maintenance cost" category takes into consideration both maintenance and operation costs.

The rating for “aesthetic fit” is an attempt to score the ease of integrating a certain design into the existing fabric and character of the landscape at a specific project location. It also considers the potential for providing additional recreational and educational benefits. For example, if a practice design has little to no adverse impacts on the original (pre-installation) look and feel of a site, it is considered to have a “moderate” aesthetic fit. If aesthetics are enhanced or degraded, the rating changes to “high” or “low”, respectively.

Table 11. Qualitative summary of stormwater BMP system capabilities, suitability for Galveston Island locations, cost, and authenticity

BMP System	Runoff Erosion Control	Runoff Quantity Control	Runoff Quality Control	Aesthetic Fit	Galveston Suitability	Construction Cost	Maintenance Cost
Infiltration basins	Moderate	High	High	High	E, W	Moderate	Low
Infiltration trenches	High	High	Moderate	High	E, C, W	Moderate	Moderate
Pervious pavement	High	High	Moderate	High	E, C, W	Moderate	Moderate
Dry detention basins	High	Low	Low	High	W	High	Moderate
Subsurface detention	High	Low	High	Moderate	E, W	Moderate	High
Retention basins	High	Moderate	Moderate	High	W	Moderate	Moderate
Retention berms	High	Low	Moderate	Moderate	E, W	Low	Low
Dune infiltration systems	High	High	Moderate	Moderate	E, C, W	Moderate	Low
Bioinfiltration / bioretention	Moderate	High	High	High	E, W	Moderate	Low
Vegetated swales	High	Low	Moderate	High	E, C, W	Moderate	Low
Vegetated filter strips	High	Low	High	High	E, C, W	Low	Low
Gravity drainage system	Moderate	High	Moderate	Moderate	E, C	Moderate	Low
Beach drainage systems	Moderate	High	Moderate	Moderate	E, C	Moderate	Low
Pressure equalizing modules	Moderate	High	Moderate	Moderate	E, C, W	Low	Low
Rock and timber revetments	High	Low	N/A	Low	E, C, W	Low	Moderate
Gabion revetments	High	Low	N/A	Low	E, C, W	Low	Moderate
Bio-thatching	High	Low	Low	Moderate	E, C, W	Low	Low

The category "Galveston suitability" assesses whether a specific BMP design could be a viable option for the three evaluated project beach sites in this study (E: east, C: central, W: west). The information on beach configuration and runoff issues identified for each site (Chapter 1), runoff catchment characteristics (Chapter 2), and primary functions and advantages/limitations of the individual BMP options (Chapter 3) were considered. This feasibility assessment is intended to provide an overview and very general guidance on individual design options to be used in further stakeholder discussions on best strategies to deal with runoff issues on Galveston Island. The main purpose is to point out options for mitigation strategies and design principles, as well as advantages and limitations of each stormwater management practice. Future more detailed site-specific analyses including regional meteorological conditions, as well as technical and economic feasibility need to be conducted prior to finalizing the optimal runoff mitigation strategy for each location. In the following section, some further thoughts on potential best options for each site are shared.

5.2 Discussion of Site-Specific Best Options

The east Galveston Island project site of Stewart Beach has experienced frequent flooding of beach parking areas and has seen development of significant scour channels even after moderate rainfall events. The naturally low infiltration capacity of the fine sediments coupled with the compaction effects from vehicle traffic in beach parking areas encourages surface runoff flows during rain events. Fronting beaches are directly connected with the compacted, bare soil surface of the parking lots and/or sparsely vegetated sand dunes. The regional runoff catchment analysis revealed a complex stream network where high-order runoff stream channels are easily created on the flat parking lot and backshore areas, confirming the vulnerability of this site to runoff scour channel creation and chronic/nuisance flooding.

A potential runoff management strategy for these east Galveston beach sites should primarily focus on reducing the volume of surface water and improving the drainage capacity of the soil. Recommended BMP applications include passive drainage systems that provide a permeable substrate layer or underground conveyance pipes through which the surface water can be infiltrated into the sublayer and then either conveyed offshore as an internal flow driven by gravity or mechanically pumped out to discharge at a desired location (e.g., offshore discharge point or inland for reuse in saltwater-tolerant systems). Groundwater conveyance can be promoted by reducing the watertable on the upper beach via installation of vertical drainage systems (e.g., PEM) to reduce the surface water quantity and reduce the drying time of the surface soils. Small-scale (bio-) infiltration systems could be installed as part of paved parking lots and along upland and overland roadways. Sidewalks incorporating pervious pavement and subsurface detention or infiltration systems could be included as an additional feature to reduce runoff discharge to the beach. Vegetated swales or filter strips can easily be implemented to disconnect and filter the overland runoff before reaching the backshore areas. Existing vegetated sandy areas affected by runoff flows could be protected via armoring and flow diversion systems.

The nourished beaches in front of the Galveston Seawall serve as a receiving basin for the rainwater runoff from directly connected impervious upland surfaces. At certain locations, these beaches have experienced the formation of significant backshore scour channels during rain events. A portion near 53rd Street (in front of The San Luis Resort) was evaluated in this study as a central Galveston Island project site example. The regional hydrology analysis confirmed the great potential for high-velocity runoff discharge across the abrupt surface drop at the seawall. In particular, the short travel times and low retention capability can lead to scour channel formation. This can jeopardize the long-term stability of the seawall and the safety of beach users in addition to having detrimental effects on regional aquatic habitats. Significant amounts of suspended solids and pollutants carried by the initial flush of the surface runoff can contaminate nearshore waters and beach sediment.

Therefore, the runoff management strategy for central Galveston Island project sites, such as in front of 53rd Street, has to focus on measures that disconnect, interrupt, and filter the runoff flow coming from the impervious surface of Seawall Boulevard and connected inland building areas before its outfall at the edge of the seawall. Recommended BMP applications include pervious pavement or infiltration trenches and vegetated swale and filter systems that can be installed alongside the inland roadways and sidewalks. On the beach along the seawall base, armoring and rough ramp systems can be introduced to provide protection against runoff scour formation and sediment washout at the outfall points of overland flows. Particularly, dune infiltration systems providing comprehensive surface water infiltration, retentive grading, filtering (via sand and vegetation), and subsurface detention, as well as armoring and diversion of flows impinging on the seawall base can be an effective measure to prevent runoff erosion on the beach. Passive beach dewatering system such as GDS and BDS may be installed in conjunction with upstream rainwater runoff treatment/collection systems so the overland discharge to the beach can be received and partially conveyed through the permeable layer within the system before causing severe beach erosion and scarping. The new “soft” engineering technique of PEM can also be considered to aid beach dewatering and drying of the surface soil.

The beaches located on the west portion of Galveston Island have a very flat topography and are backed by residential neighborhoods. A section of Pirates Beach was used as an example in the present study. Frequent ponding/flooding problems even after moderate rainfall events are common there according to local residents. Some drainage pipes installed to discharge overflow water through the dune line to the open beach have promoted berm and dune erosion and have created large scour holes near the beachside pipe outfalls. Repeatedly, structural pipe failures due to the loss of the supporting sediment at the outfall and frequent clogging of the pipe inlet by debris and sediment scarped from roadside ditches were observed during large runoff events. The synthetic stream network developed for the Pirates Beach project site indicated that the surface runoff from the northern and southern part of the residential areas were converging on the street located in the center of the neighborhood (Grand Terre) before being conveyed to the beach for discharge. Therefore, significant localized beach erosion, sand loss, and high concentration of contaminant in the backshore areas of the beach at the outfall of the high-order, high-velocity, channelized runoff streams occurred.

Recommended BMP applications for the west Galveston Island project site include (bio-) infiltration systems that are effective in transforming surface flows into substrate flows. This reduces the surface discharge quantity. Vegetated swales and filter strips can effectively control both the runoff flow velocities (travel time, t_c) and runoff water quality. Runoff scour impacts at the beachside outfall sites can be alleviated by flow diversion, armoring, and bio-thatching systems. Effective landscaping and implementation of small-scale BMP designs by local residents can have significant effects on controlling the peak runoff rate at the inlet and outlet of the existing discharge pipes. Therefore, installation of pervious pavement on the driveways and rain gardens on individual lots in addition to retentive grading across the roadside ditches and construction of retention/detention ponds should be considered. In addition, implementation of small-scale runoff control devices such as rain barrels and green roof systems by individual households can further improve the situation using non-structural BMPs. At the beachside drainage pipe outlets, armoring and diversion systems can be installed to provide protection against scour from concentrated runoff. Furthermore, beaches at this location are relatively narrow, which allows tidal fluctuations in water level to exacerbate the erosion problem at the pipe outfall points and even allows for interaction of runoff and wave or tidal action. Therefore, implementing innovative “soft” engineering techniques such as PEMs should be considered to aid beach stabilization by enhancing dewatering and drying mechanisms for the surface soil.

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APPENDIX

Table A- 1. Peak runoff rates and potential runoff flooding rates estimated at varying rainfall intensities for the Stewart Beach site.

	Q = CIA											
	2-year		5-year		10-year		25-year		50-year		100-year	
	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr
E05	0.5	1.0	0.7	1.3	0.8	1.5	0.9	1.7	1.1	2.0	1.2	2.3
E06	0.4	1.4	0.5	1.8	0.6	2.0	0.7	2.4	0.8	2.7	1.0	3.2
E20	1.4	1.9	1.8	2.5	2.1	2.9	2.5	3.4	2.8	3.8	3.2	4.4
E25	0.2	4.4	0.3	5.4	0.3	6.1	0.4	7.0	0.4	7.9	0.5	8.9
E29	0.4	4.2	0.5	5.1	0.5	5.8	0.6	6.7	0.7	7.5	0.8	8.5
E33	1.1	4.3	1.4	5.3	1.6	6.0	1.8	6.9	2.1	7.8	2.3	8.7
E35	0.4	4.8	0.5	5.9	0.6	6.6	0.6	7.6	0.7	8.5	0.8	9.6
E36	0.6	5.3	0.8	6.3	0.9	7.1	1.0	8.1	1.1	9.1	1.2	10.1

Table A- 2. Potential hydraulic power associated with peak runoff rates discharging at the downstream pour points of respective drainages estimated at varying rainfall intensities for the Stewart Beach site.

	$P = \rho g Q H_{100\%}$											
	2-year		5-year		10-year		25-year		50-year		100-year	
	kW	hp	kW	hp	kW	hp	kW	hp	kW	hp	kW	hp
E05	44	60	57	76	66	89	78	105	89	119	102	137
E06	37	50	48	64	55	74	65	88	74	100	86	115
E20	139	186	178	238	206	276	243	325	276	371	317	426
E25	11	15	14	19	16	21	18	24	20	27	23	31
E29	27	36	33	44	37	50	43	58	48	65	54	73
E33	90	121	111	149	125	168	145	194	163	219	184	246
E35	25	33	30	40	34	45	39	52	44	59	49	66
E36	50	67	61	81	68	91	78	104	87	117	97	130

Table A- 3. Peak runoff rates and potential runoff flooding rates estimated at varying rainfall intensities for the central Galveston project site.

	Q = CIA											
	2-year		5-year		10-year		25-year		50-year		100-year	
	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr
C01	0.1	5.7	0.1	6.7	0.1	7.5	0.2	8.5	0.2	9.5	0.2	10.5
C02	0.2	5.0	0.2	6.0	0.2	6.7	0.2	7.6	0.3	8.6	0.3	9.5
C03	0.1	6.0	0.1	7.1	0.1	7.9	0.1	9.0	0.1	10.1	0.2	11.1
C04	0.2	5.2	0.2	6.2	0.3	6.9	0.3	7.9	0.3	8.9	0.4	9.9
C05	0.2	5.7	0.3	6.9	0.3	7.7	0.3	8.7	0.4	9.8	0.4	10.8
C06	0.4	4.4	0.5	5.4	0.5	6.0	0.6	6.9	0.7	7.7	0.7	8.6
C07	0.1	5.6	0.1	6.7	0.1	7.5	0.2	8.6	0.2	9.6	0.2	10.7
C08	0.3	5.6	0.3	6.7	0.4	7.4	0.4	8.5	0.5	9.5	0.5	10.5
C09	0.2	4.0	0.3	4.9	0.3	5.5	0.4	6.3	0.4	7.2	0.4	8.0
C10	0.1	3.6	0.1	4.5	0.1	5.1	0.1	5.8	0.1	6.6	0.1	7.4
C11	0.2	3.6	0.2	4.4	0.2	5.0	0.2	5.7	0.3	6.5	0.3	7.3

Table A- 4. Potential hydraulic power associated with peak runoff rates discharging at the downstream pour points of respective drainages estimated at varying rainfall intensities for the central Galveston project site.

	$P = \rho g Q H_{100\%}$											
	2-year		5-year		10-year		25-year		50-year		100-year	
	kW	hp	kW	hp	kW	hp	kW	hp	kW	hp	kW	hp
C01	4	5	5	6	5	7	6	8	7	9	7	10
C02	7	9	8	11	9	12	10	14	11	15	13	17
C03	4	6	5	7	6	8	7	9	7	10	8	11
C04	18	24	22	29	24	32	28	37	31	42	34	46
C05	28	38	34	45	38	51	43	57	48	64	53	71
C06	43	58	52	70	58	78	66	89	75	100	83	112
C07	4	6	5	7	6	7	6	8	7	10	8	11
C08	16	22	19	26	22	29	25	33	28	37	31	41
C09	12	16	15	20	17	23	19	26	22	29	24	33
C10	3	4	4	6	5	6	5	7	6	8	7	9
C11	9	12	11	15	13	17	14	19	16	22	18	24

Table A- 5. Peak runoff rates and potential runoff flooding rates estimated at varying rainfall intensities for the Pirates Beach site.

	Q = CIA (in./hr)											
	2-year		5-year		10-year		25-year		50-year		100-year	
	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr	m^3/s	in./hr
W01	0.1	5.5	0.1	6.5	0.1	7.2	0.2	8.2	0.2	9.2	0.2	10.2
W02	2.4	3.0	2.9	3.8	3.4	4.3	3.9	5.0	4.4	5.6	5.0	6.4
W03	0.3	4.5	0.3	5.4	0.4	6.0	0.4	6.8	0.5	7.7	0.5	8.5
W04	3.1	3.3	3.8	4.1	4.3	4.7	5.0	5.4	5.7	6.2	6.4	7.0

Table A- 6. Potential hydraulic power associated with peak runoff rates discharging at the downstream pour points of respective drainages estimated at varying rainfall intensities for the Pirates Beach site.

	$P = \rho g Q H_{100\%}$											
	2-year		5-year		10-year		25-year		50-year		100-year	
	kW	hp	kW	hp	kW	hp	kW	hp	kW	hp	kW	hp
W01	1	2	2	2	2	3	2	3	2	3	3	4
W02	206	276	257	344	293	393	341	457	386	517	437	586
W03	7	10	9	12	10	13	11	15	12	17	14	18
W04	291	391	362	485	412	552	478	641	540	725	611	819