DWR – NPDES-SOP – G – 16 –Erosion Prevention and Sediment Control Handbook - 01092026
Erosion Prevention and Sediment Control Handbook

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SIGNATURES:

April Grippo (Dec 30, 2025 11:47:47 CST)
April Grippo,
Division Director DWR

Hannah Riether Digitally signed by Hannah Riether Date: 2025.12.30 10:43:13 -06'00'
Hannah Riether, PE
Preparer

Karina Bynum Digitally signed by Karina Bynum Date: 2025.12.30 10:48:54 -06'00'
Karina Bynum, PhD, PE
Reviewer

PURPOSE

The Erosion Prevention and Sediment Control (EPSC) Handbook serves as the primary technical reference for the design and implementation of EPSC measures to limit the export of sediment to surface waters in Tennessee. The last edition of the Handbook was completed in August 2012. This revision updates various technical specifications for EPSC measures based on the most current research.

REVISION HISTORY TABLE

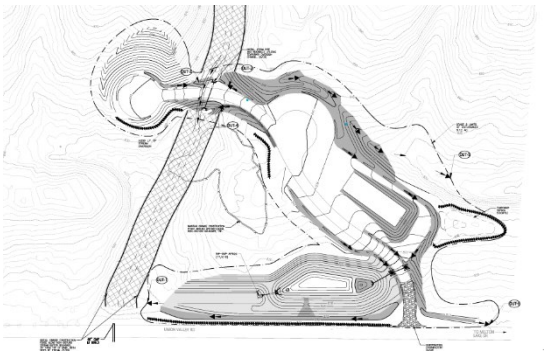
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TENNESSEE
EROSION PREVENTION AND SEDIMENT CONTROL HANDBOOK

A Technical Guide to Construction Best Management Practices


DUCK RIVER, COLUMBIA, TN







TN Department of Environment & Conservation

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**Names are listed alphabetically by last name in each section.*

Primary Authors

Hannah Riether, P.E., TDEC-DWR
Ian Simpson, PhD, E.I., UTK – TNWRRC

Contributing Authors

Jon Hathaway, PhD, P.E., UTK – TNWRRC and Dep. of Civil and Environmental Engineering
J. Blake Whitman, PhD, P.E., CPESC, UTK – Dep. of Biosystems Engineering and Soil Science

Chief Technical Advisors

Karina Bynum, PhD, P.E., TDEC-DWR
John Schwartz, PhD, P.E., UTK – TNWRRC and Dep. of Civil and Environmental Engineering

Technical Committee

Wesley Apple, P.E., TDOT
Dale Binder, Metro Water Services
Josh Burnette, P.E., Renaissance Design Studio, LLC (RDS)
Steve Casey, P.E., CPESC, Civil & Environmental Consultants, Inc. (CEC)
Harris Darby, P.E., Virginia Department of Transportation (VDOT)
David Flynn, P.E., SSOE
Joshua Frerichs, P.E., CFM, City of Knoxville
Matt Glenn, City of Chattanooga
Kris Gordon, P.E., West Tennessee River Basin Authority (WTRBA)
Chip Hannah, TDOT
Shawn Herman, Metro Water Services
Doug Noonan, City of Franklin
Greg Russell, TDOT
Carma Smith, TDOT
Will Stallard, E.I., City of Kingsport
Robbie Stephens, P.E., TDOT
Trent Thomas, TDOT

Special Thanks to Laura Chandler, P.E. (TDOT), Michael Hunt (Metro Water Services), and Jeff Willoughby (City of Franklin) for providing their staff's time and resources. As well as Crystal Bishop [Tennessee Valley Authority (TVA)], John Buchanan, PhD, P.E. (UTK), William Chevales (TVA), and Tim Gangaware (UTK) for their support.

Additional Tennessee Department of Environment and Conservation -Division of Water Resources Reviewers
Joellyn Brazile
Eddie Gordon
Angela Jones, P.E., CPM
Ann Morbitt
Ariel Wessel-Fuss, E.I.

Ad hoc Reviewers
John Chlarson, P.E., CPM, University of Tennessee Municipal Technical Advisory Service
Caitlan Elam, TDEC-Natural Heritage
Jason Miller, Tennessee Wildlife Resources Agency (TWRA)
Ross Rogers, TDEC-DWR
Crittenden Stender, G.I.T., TDEC-DWR
Louan Tillman, Integrated Technologies LLC

Support Team
Carley Ferguson, TDEC-DWR
Regan McGahen, TDEC-DWR
And TDEC-DWR Engineering Services Unit staff

Example Drawings
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Seed Mix Appendix Committee:
Todd Crabtree, TDEC-Natural Heritage
Adam Dattilo, TDEC-Natural Heritage
Lauren Deem, Profile Products LLC
Evelyn Diorio, TDOT
Becky Grubbs-Bowling, PhD, UTK
Jeremy Hamlington, Roundstone Native Seed LLC
Chip Hannah, TDOT
Nathaniel Hicklin, Natural Resource Conservation Service (NRCS)
Robert Hoffman, Roundstone Native Seed LLC
Andrea Ludwig, PhD, UTK
Michael McCord, TWRA
Mike Ross, UTK
John Seymour, Roundstone Native Seed LLC
Eric Shiflett, TDEC-DWR
Joe Stephens, Stephens Supply Co.
Virginia Sykes, PhD, UTK
Marc Theisen, Profile Products LLC
Trent Thomas, TDOT
Tyler Wayland, Roundstone Native Seed LLC
Shawn Wurst, TDOT

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Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ARAP	Aquatic Resource Alteration Permit
ARC	Antecedent Runoff Condition
ASTM	American Society for Testing and Materials
BMP	Best Management Practice
CAP	Corrective Action Plan
CFR	Code of Federal Regulations
CGP	Construction General Permit
CN	Curve Number
CWA	Clean Water Act
DOT	Department of Transportation
ECB	Erosion control blanket
EPSC	Erosion Prevention and Sediment Control
ETW	Exceptional Tennessee Waters
FDA	Food and Drug Administration
FHWA	Federal Highway Administration
HDPE	High Density Polyethylene
HEC	Hydraulic Engineering Circular
HECP	Hydraulic Erosion Control Product
HSG	Hydrologic Soil Group
MS4	Municipal Separate Storm Sewer System
MUSLE	Modified Universal Soil Loss Equation
NEH	National Engineering Handbook
NHD	National Hydrography Database
NOAA	National Oceanic and Atmospheric Administration
NOC	Notice of Coverage
NOI	Notice of Intent
NOT	Notice of Termination
NOV	Notice of Violation
NPDES	National Pollutant Discharge Elimination System
NRDA	Natural Resource Damage Assessment
NRCS	Natural Resource Conservation Service

OCF	Open Channel Flow
OHSA	Occupational Health and Safety Administration
SCS	Soil Conservation Service
ONRW	Outstanding National Resource Waters
PAM	Polyacrylamide
QHP	Qualified Hydrologic Professional
QLP	Qualifying Local Program
QPL	Qualified Product List
RECP	Rolled Erosion Control Product
RUSLE2	Revised Universal Soil Loss Equation 2
SCF	Shallow Concentrated Flow
SCM	Stormwater Control Measure
SF	Sheet Flow
SWPPP	Stormwater Pollution Prevention Plan
T_C	Time of Concentration
TDEC	Tennessee Department of Environment and Conservation
TDEC-DWR	Tennessee Department of Environment and Conservation Division of Water Resources
TDEC-EFO	Tennessee Department of Environment and Conservation Environmental Field Office
TDOT	Tennessee Department of Transportation
TMDL	Total Maximum Daily Load
TMSP	Tennessee Multi-Sector General Permit for the Discharge of Stormwater from an Industrial Activity
TNWRRC	Tennessee Water Resources Research Center
TRM	Turf Reinforced Mat
TT	Travel Time
TVA	Tennessee Valley Authority
TWQCA	Tennessee Water Quality Control Act
UIC	Underground Injection Control
USACE	United States Army Corp of Engineers
USDA	United States Department of Agriculture
USDA-ARS	United States Department of Agriculture Agricultural Research Service

(US)EPA	(United States) Environmental Protection Agency
USGS	United States Geological Survey
USLE	Universal Soil Loss Equation

Chapter 1 - Introduction

1.1 Purpose

This *Erosion Prevention and Sediment Control (EPSC) Handbook* has been developed to provide comprehensive and standardized EPSC measures for use on construction sites to limit the export of sediment to surface waters in Tennessee. The Tennessee Construction General Permit [CGP; (TDEC)] is predominantly focused on pollution caused by exposed and destabilized soils; therefore, this handbook is designed to provide information on how to prevent, minimize, and treat sediment-laden stormwater discharges during construction and site development. This handbook serves as the primary reference for the design and implementation of all such measures and practices under the CGP. This update of the handbook serves to be a more technical reference than the previous version (2012) by streamlining text and logical flow, expanding the list of EPSC measures, and updating various technical specifications for EPSC measures.

The first and most important step in erosion prevention and sediment control is minimizing soil disturbance and preserving the existing vegetative cover. Well-planned projects avoid and minimize adverse impacts, and where disturbance is necessary, the least impactful construction processes are chosen. Final stabilization should be implemented as soon as possible.

Site specific stormwater pollution prevention plans (SWPPPs) are required in order to receive coverage under the CGP. The SWPPP lays out the steps and techniques the operator will use to reduce pollutants in stormwater runoff leaving their construction site. The EPSC plans are design drawings that show the placement, size, and details of EPSC measures. This handbook is to be used as design guidance for the EPSC measures. More details of what is required in the SWPPP can be found in the CGP.

1.2 Erosion and Sediment Impairments to Water Quality

Erosion is the process by which land surfaces are worn away by the impacts of raindrops or by the shear forces of stormwater. Stormwater is runoff generated during rainfall and snowmelt events that flows over land (i.e., does not infiltrate into the ground) or impervious surfaces such as paved streets, parking lots, and building rooftops. Erosion is a natural process that, in undisturbed conditions, occurs at extremely slow rates depending on rainfall, landscape characteristics, soil properties, and vegetation. However, human land-disturbing activities accelerate erosion rates several orders of magnitude from average rates of less than one ton per acre per year in forested or undisturbed settings to seven tons per

acre per year in developed settings and to approximately 200 tons per acre per year on construction sites (Borrelli et al., 2021; Nearing et al., 2017).

Eroded soils can be transported short distances when soil particles are detached by raindrop impact and longer distances when the detached particles are transported by stormwater. Additional soil particles can be detached and transported if the velocity of overland flow is increased due to intense storms, steep slopes, compacted soils, impervious areas, etc. If not managed properly, overland flow will begin to concentrate and erode small channels or rills and will eventually form larger channels or gullies, which, in turn, allow for larger shear forces and continued erosion.

Eroded soils travel in overland flow and are eventually discharged to receiving bodies of water. Once these soils enter larger bodies of water and the velocity of the water slows, the soils begin to settle, a process known as sedimentation. Larger, coarser particles, such as gravels and sands, will settle much quicker than medium-sized particles (silts) and fine particles (clays). Siltation is a condition where the natural bottom of a stream is covered in silts and clays. Sediment-laden discharges from construction activities, if not managed properly, lead to adverse water quality, food web alterations, biologic suffocation, stream geomorphic changes, increased flooding, restrictions on navigation, and turbid waters, which impact fish feeding and plant photosynthesis. Sediment is one of the top three causes of impairment within the state of Tennessee and the United States' surface waters. As such, there are 16 billion dollars in environmental damages due to sedimentation annually (Smith, 2018). Communities, construction and development companies, and industries help protect Tennessee’s water resources by using stormwater controls, known as best management practices (BMPs), to limit sediment pollution and other pollutants that preferentially bind to sediments. Herein, BMPs used specifically in construction applications are referred to as EPSC measures.

1.3 Regulations

The National Pollutant Discharge Elimination System [NPDES; (CFR122)] program regulates stormwater discharges under the Clean Water Act (CWA) from three potential sources: municipal separate storm sewer systems (MS4s), industrial activities, and construction activities. Operators of these sources might be required to obtain coverage under the NPDES permit before they can discharge stormwater. The NPDES permitting mechanism is designed to prevent stormwater runoff from washing toxic or harmful pollutants into surface waters or infiltrating into groundwater sources.

The U.S. Environmental Protection Agency (EPA) has authorized the Tennessee Department of Environment and Conservation, Division of Water Resources (TDEC-DWR) to implement the NPDES program for the state of Tennessee, thereby regulating point and non-point

sources of pollution. Construction stormwater is regulated under Tennessee's *General NPDES Permit for Discharges of Stormwater Associated with Construction Activities*, commonly known as the CGP (TDEC), and in some situations, under an individual permit.

CGP coverage is required when construction activities will disturb one or more acres of soil or disturb less than one acre of soil but are part of a larger common plan of development or sale that comprises at least one acre of cumulative land disturbance. CGP coverage is also required for projects of less than one acre of total land disturbance if TDEC has determined that the stormwater discharge from a site is causing, contributing to, or is likely to contribute to a violation of a state water quality standard or has determined that the stormwater discharge is, or is likely to be a significant contributor of pollutants to waters of the state. The CGP requires: (1) EPSC measures to be used during construction; (2) setbacks from waterbodies (water quality riparian buffer zones); (3) a site specific SWPPP; (4) self-monitoring site inspections; and (5) final stabilization before permit termination. Refer to the CGP for the requirements for obtaining coverage and where and how to submit the CGP Notice of Intent (NOI), SWPPP, and fee. More information can be found at the TDEC Construction Stormwater website: https://www.tn.gov/environment/permit-permits/water-permits1/npdes-permits1/npdes-stormwater-permitting-program/npdes-stormwater-construction-permit.html.

In the early 1990s and then again in 2003, the EPA finalized regulations requiring MS4 operators to meet specific criteria to obtain coverage under an NPDES permit for stormwater discharges via an MS4. Check with the local jurisdiction early in the planning process to understand local construction stormwater requirements and how they may affect the site specific SWPPP and EPSC plans. Both local and state requirements must be met. A list of Tennessee's designated MS4 programs can be found on TDEC's website: https://www.tn.gov/environment/permit-permits/water-permits1/npdes-permits1/npdes-stormwater-permitting-program/npdes-municipal-separate-storm-sewer-system--ms4--program.html.

The CGP requires a higher standard for design of EPSC measures, water quality riparian buffer zones (setbacks), and EPSC measures for construction stormwater discharges to Exceptional Tennessee Waters (ETWs) and waterbodies that have unavailable parameters for siltation. ETWs are surface waters designated by the TDEC-DWR as having the characteristics set forth at Tennessee Rules, Chapter 0400-40-03-.06(4). Waters with unavailable parameters is any segment of surface waters that has been identified by TDEC-DWR as failing to support one or more classified uses. The term unavailable parameters is not the same as unassessed. If a stream has unavailable parameters, the stream has been assessed and cannot receive any more of the specified pollutant. This terminology has replaced what used to be termed an impaired stream. Therefore, the impairments would be the unavailable parameters of

that stream. For the purpose of the CGP, the pollutant of concern is siltation. Thus, if there was a stream that had unavailable parameters for siltation, this would indicate the stream is unable to take in any more sediment; it is already impaired by sediment, so the stream must have more conservative EPSC measures in place to ensure no further sediment pollution to the stream. Waters with unavailable parameters that are impaired for reasons other than sedimentation/siltation (such as nutrients, *Escherichia coli*, physical alterations, etc.) are not required to meet the higher design standards unless sedimentation/siltation is also listed as a cause of impairment. To determine the classification of receiving water bodies, utilize the TDEC-DWR construction stormwater permitting map viewer found at https://tdeconline.tn.gov/dwrcgp/, and the list of known ETWs can be found at https://dataviewers.tdec.tn.gov/dataviewers/f?p=2005:34304:3123833 63635:.

Some MS4 programs may require a more conservative design for waterbodies that have unavailable parameters for other contaminants, even if siltation is not listed. The more stringent requirements between the CGP and local ordinances must be followed. If a construction area is within MS4 jurisdiction and coverage under the CGP is required from the state, a land disturbance permit is also required from the MS4. Often, the MS4 will have additional regulations (beyond the CGP), such as water quantity, emphasizing the importance of coordinating with the local jurisdiction. Obtaining coverage under the CGP does not insinuate coverage from the MS4 unless it is a Qualified Local Program (QLP) and has been given delegated authority from the state to provide CGP coverage on behalf of TDEC.

A total maximum daily load (TMDL) is the sum of the individual wasteload allocations for point sources and load allocations for nonpoint sources and natural background (40 CFR 130.2(I)). TMDLs are a study that quantifies the amount of a pollutant in a stream, identifies the sources of the pollutant, and recommends regulatory or other actions that may be needed in order for the stream to recover from pollution. Coverage under the CGP can only be obtained if the SWPPP incorporates measures or controls consistent with the assumptions and requirements of the TMDL (if the TMDL is for sediment or urban runoff).

Tennessee's water quality is regulated through the Water Quality Control Act, T.C.A., §69-3-101, et seq., and Chapters 0400-40-03 (General Water Quality Criteria) and Chapter 0400-40-04 (Use Classifications for Surface Water) of Rules of the TDEC-DWR. These regulations outline designated uses for streams as well as the quality of all types of discharges. According to the state's antidegradation policy (Chapter 0400-40-06 of Rule of the TDEC-DWR), new discharges must not alter or degrade the quality of the receiving waterbody. If the quality of water can support either the propagation of fish, shellfish, and wildlife or recreational activities, then the water quality must be maintained and protected unless TDEC-DWR finds

that the degradation of the water quality is necessary to accommodate important economic or social development in the area.

1.3.1 Water Resources Inventory

An inventory of all water resources (streams, rivers, lakes, wetlands, etc.) must be completed and clearly shown on the EPSC plans. A formal process for determining jurisdictional streams and wet weather conveyances in Tennessee is called hydrologic determination (Rule 0400-4-03 as provided for in Public Chapter 464 of 2009). A hydrological determination must be completed by a certified TN-Qualified Hydrologic Professional (QHP) and submitted to TDEC-DWR for concurrence of the findings on site. Wetlands must be identified and delineated by wetland professionals. Wetland delineation must be submitted to the TDEC-DWR for concurrence, and the confirmed delineation must be included in the water resources inventory. This documentation and process prevent water resources from being destroyed/altered without permitting or mitigation. The water resource inventory on the EPSC plans also indicates what water resources are on the site, where the riparian buffer zones will be placed/conserved, what may be impacted due to construction, and if any additional permits may be required.

1.3.2 Other Permits and Regulations

A person who wishes to make an alteration to a stream, river, lake, or wetland must first obtain a water quality permit. Physical alterations to properties of waters or the state require an Aquatic Resource Alteration Permit (ARAP; Chapter 0400-40-04 Aquatic Resource Alteration). Depending on the extent of the impact on the water resource, these permitted activities fall into two categories: those that can be authorized under a general permit and those that require an individual ARAP. Many, if not most, of the activities requiring an ARAP also require coverage under the CGP. The majority of alterations that require an ARAP will also require a U.S. Army Corps of Engineers (USACE) Section 404 permit and possibly a Section 10 permit for projects that include the discharge of dredged or fill material into waters of the U.S., including wetlands. USACE should be contacted directly to determine if acquiring these permits is required for the proposed site. When a 404 certification is required from USACE, a 401 certification must first be obtained from the TDEC. General ARAPs provide a streamlined means for the TDEC-DWR to approve activities considered to result in minor impacts. Some General ARAPs require prior TDEC notification or approval before beginning the activity; others only require that the activity be conducted in accordance with the conditions of the General ARAP. Individual Permits are required when an activity does not qualify for coverage under a General Permit. Only projects that demonstrate the least impactful practicable alternative to the proposed activity will be eligible for coverage by an Individual Permit. More information on each permit type can be found at TDEC's ARAP

website: https://www.tn.gov/environment/permit-permits/water-permits1/aquatic-resource-alteration-permit--arap-.html.

Underground injection control (UIC) applications must be submitted to TDEC-DWR, Drinking Water Unit, when emplacing any fluids into the subsurface via a discrete point of injection (e.g., sinkhole, rock crevice/crack, dug hole, drilled shaft, etc.). Wells are divided into six classifications depending on their use and risk for groundwater contamination, wherein the construction of a well used for stormwater management would be considered a Class V injection well. More information regarding UIC and well classification can be found at TDEC's UIC website: https://www.tn.gov/environment/program-areas/wr-water-resources/water-quality/underground-injection-control--uic-.html.

TDEC-DWR Safe Dams Program is responsible for conducting inspections, maintaining an inventory of dams within the state, reviewing plans, and issuing certifications of operation, alteration, and construction of dam projects. With regards to this document, an EPSC measure would be considered a dam, and therefore must be regulated, if the structure impounds 30 acre-feet or more of water or is, at minimum, 20 feet in height (measured from the downstream toe elevation to the low point of the dam crest). Under rule 0400-45-02, "any such barrier which is or will be less than six (6) feet in height, regardless of storage capacity, or which has or will have a maximum storage capacity not in excess of fifteen (15) acre-feet, regardless of height, shall not be considered a dam." All non-federal dams are required to have a certificate of approval from the Commissioner to construct, alter, or operate an impoundment. Non-federal dams may also require other environmental permits, such as an ARAP or a USACE 404 Permit, even though the dam may not be subject to the Safe Dams Act. More information can be found at TDEC's safe dams website: https://www.tn.gov/environment/program-areas/wr-water-resources/water-quality/safe-dams-program.html.

The Tennessee Valley Authority (TVA) is a federal agency serving several purposes: to improve navigability and provide for flood control of the Tennessee River; to provide for reforestation and the proper use of marginal lands in the Tennessee Valley; and to provide for the agricultural and industrial development of the valley. The TVA Act, Section 26A, requires that TVA approval be obtained before carrying out any construction activities that affect navigation, flood control, or public lands along the shoreline of TVA reservoirs, in the Tennessee River, or in Tennessee River tributaries. TVA 26A is designed to ensure that construction along the shoreline does not have a negative effect on the agency's management of the river system. Permit approvals for construction under Section 26A are considered federal actions and are, therefore, subject to the National Environmental Policy Act and other federal laws. Typical structures and projects that require TVA Section 26A

approval include boat docks, piers, boat ramps, bridges, culverts, commercial marinas, barge terminals and mooring cells, water intake and sewage outfalls, and fill or construction within the floodplain. For more information on TVA 26A permits, refer to their website: https://www.tva.com/environment/shoreline-construction-permits.

The NPDES program regulates stormwater discharges from certain *industrial* activities. Operators of these sources might be required to obtain coverage under an NPDES permit before they can discharge stormwater. Industrial stormwater discharges are primarily regulated under the *NPDES Tennessee Stormwater Multi-Sector General Permit for Industrial Activities* (TMSP) and, in some situations, under an NPDES individual permit. Requirements of the TMSP are separated by the standard industrial classification or SIC code. For more information, refer to TDEC's website: https://www.tn.gov/environment/permit-permits/water-permits1/npdes-permits1/npdes-stormwater-permitting-program/npdes-industrial-stormwater-general-permit.html.

The Endangered Species Act, U.S. Fish and Wildlife Service, and National Invasive Species Council all impose contingencies for construction if the construction would disrupt an ecosystem that contains threatened and endangered species of fish, wildlife, and plants; does not comply with the Migratory Bird Treaty Act, Marine Mammal Protection, or harms a species on the list of federally endangered or threatened species; and if construction has the potential to cause or promote the introduction of invasive species, respectively. These three acts impose contingencies for the performance of construction; however, they do not impact the process of obtaining a CGP. Refer to the National Heritage Inventory Program's website for full compliance with the Endangered Species Act (https://www.tn.gov/environment-/program-areas/na-natural-areas/na-natural-heritage-inventory-program.html); the Fish and Wildlife Service's website for full compliance with their stipulations (www.fws.gov/); and the National Invasive Species Council's website to prevent the introduction of new species (https://www.usda.gov/topics/invasive-species).

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Chapter 2 - Stormwater Management

2.1 Hydrology and Drainage Calculations

There are a variety of BMPs that can be designed to capture, store, and treat stormwater runoff from construction sites. Commonly, EPSC measures and BMPs are categorized by their purpose and classified as: stabilization, pollution prevention, erosion prevention, runoff velocity control, sediment control, or stream protection practices, or a combination of the categories. Further, EPSC measures can be structural (i.e., physical structures with engineering design) or nonstructural (i.e., implemented methodologies through planning and policies) practices. Regardless of their classification, many structural EPSC measures are sized by their projected hydrologic or hydraulic capacity. In some scenarios, sizing, or design of EPSC measures may be governed by regulations, hydrologic/hydraulic calculations, or a combination of the two. The following information is presented under the assumption that the designer is proficient with watershed delineations, hydrology, hydraulics, and stormwater runoff. Detailed information may be located in the *Tennessee Department of Transportation Drainage Manual* (TDOT) and the *National Engineering Handbook* [NEH (NRCS, 1967–2019)].

Estimating the peak flow and volume of runoff from a construction site assists in calculating erosion potential, flow routing, and required sediment treatment from EPSC measures. Estimating the maximum flow rate leaving a construction site will guide the sizing of inlets, outlets, and emergency spillways of the EPSC measures. Therefore, accurate, predictive methodologies for both runoff volume and peak flow rates are critically important in the sizing and design of such measures. Hydrologic procedures for the Natural Resources Conservation Service (NRCS) Technical Release 55 (TR-55) method are recommended for design and outlined in the following subsections; however, other hydrologic methodologies such as the Rational method (Schueler, 1987), Small Storm Hydrology method (Pitt, 1994), or Green-Ampt method (Green & Ampt, 1911), may be suitable alternatives in particular scenarios for hydrologic computations. For example, the Rational method may be useful for calculating peak flows (more) quickly and relating those flows to peak velocities. This approach may be of interest since peak velocities are directly related to erosive forces and eroded sediments; therefore, the Rational method is also outlined in Section 2.1.3. Common hydrologic software for the NRCS TR-55 methodology include WinTR-55, HEC-HMS, and HydroCAD® which were developed by the United States Department of Agriculture (USDA), the United States Army Corp of Engineers (USACE), and HydroCAD Software Solutions LLC. Computer based software may be a more practical approach than hand calculations due to reduced computation time and the ability to rapidly reanalyze site specific scenarios by changing key inputs.

2.1.1 Runoff Volume

The NRCS runoff curve number (CN) method is the most commonly implemented runoff volume estimation method in the United States. In this method, runoff depth (Q_{CN} [inches]) is a function of precipitation depth (P [inches]), the maximum potential retention in a watershed (S [inches]), and the initial abstractions within the watershed (I_a [inches]), provided that $P > I_a$ (Eqn 1). When $I_a \geq P$, all rainfall is captured within the watershed, and no runoff is produced.

$$Q_{CN} = \frac{(P - I_a)^2}{(P - I_a)+S} \quad \text{(Eqn 1)}$$

Maximum potential retention is an empirically derived variable (Eqn 2). Initial abstraction is also empirically derived and is assumed to have a constant, linear relationship with S (Eqn 3). This linear relationship (λ) is typically set at 0.2, meaning no runoff occurs until 20% of the watershed's ability to retain water has been filled.

$$S = \frac{1000}{CN_{comp}} - 10 \quad \text{(Eqn 2)}$$

$$I_a = \lambda \times S \quad \text{(Eqn 3)}$$

The CN_{comp} variable accounts for watershed heterogeneity in terms of land use and hydrologic soil group (HSG). More specifically, individual CNs are a function of specific land use and HSG; they can be found in tables provided in the NRCS TR-55 manual (Cronshey, 1986). When a given watershed has more than one land use or more than one HSG, individual CNs can be area weighted to obtain a composite curve number (CN_{comp}) as shown in Eqn 4, where i represents a distinct combination of land use and HSG.

$$CN_{comp} = \frac{\sum_{i=1}^{i}(CN_i \times A_i)}{\sum_{i=1}^{i} A_i} \quad \text{(Eqn 4)}$$

The CN_{comp} can be further adjusted to account for variability arising from soil moisture conditions, canopy cover density, stages of growth for vegetation, temperature, rainfall intensity, and rainfall duration. Collectively, these sources of variability are termed antecedent runoff conditions (ARC) and applying the ARC factor to the CN_{comp} is an optional process that is dependent on professional judgment. ARC is divided into three classes: class I for dry conditions, class II for average conditions, and class III for wet conditions. Some works advise converting the CN_{comp} from case II to case I or III based on previous rainfall (Ward & Trimble, 2003). Rough guidelines are that conditions may be considered dry (ARC I) if the precipitation during the previous five days was less than 0.5 inches during the dormant season or 1.4 inches during the growing season; conditions may be considered wet (ARC III) if the precipitation during the previous five days was more than 1.1 inches during the dormant season or 2.1 inches during the growing season. Tables to convert the ARC II CN_{comp} to ARC I or ARC III are provided in NEH-10 (NRCS & USDA, 2004). However, there is relatively little evidence to suggest applying ARC corrective factors is appropriate. Simpson et al. (2023)

discovered ARC corrective factors adjusted the CN_{comp} too severely therefore, professional judgement is advised when implementing ARC factors in development projects. Case III is the most conservative estimate, followed, by case II. More conservative estimates are recommended in the design process to ensure adequate storage volumes.

The overarching process for the NRCS TR-55 composite CN method for calculating runoff volume is outlined:

- Determine the area which drains to an outlet of interest (i.e., determine the watershed of interest);
- Identify and classify the specific soils, soil types, and land uses within the watershed boundary;
- Obtain CNs for each distinct combination of land use and HSG; calculate CN_{comp} (Eqn 4);
- Determine if an ARC factor should be applied;
- Calculate S and I_a (Eqns 2 & 3);
- Determine the design P value which is the site specific precipitation depth acquired from National Oceanic and Atmospheric Administration (NOAA) Atlas 14 that correlates with the design storm that can be found in the construction general permit [CGP, (TDEC)];
- Calculate Q_{CN} (Eqn 1); and
- Determine the runoff volume by multiplying Q_{CN} with the watershed area and a unit conversion factor.

Further, other CN methodologies exist beyond the composite approach. These include the distributed CN and discrete CN approach (Simpson et al., 2023) as well as modifying λ (Hawkins et al., 2009). However, when implementing a λ value other than 0.2, the alternate value cannot be simply substituted into Eqn 3 as the algebraic and statistical derivation of Eqns 1-3 result from the assumption that λ is set at 0.20 (Hawkins et al., 2008). These methodologies have proven to be more accurate than the composite method under specific watershed and rainfall conditions. While the composite CN approach is the most commonly implemented CN methodology, these other approaches may be viable alternatives, given professional judgement.

2.1.2 Time of Concentration

Estimating peak flow rates relies on a timing component. This timing component is known as the time of concentration (T_c) which can be defined as the time it takes for water to travel from the hydrologically most distant point in a watershed to the watershed outlet. Therefore, the peak flow rate will occur during the intensity of a storm over a duration that matches the

T_c. Estimating T_c can be influenced by a number of variables including watershed surface roughness, shape, size, and slope as well as rainfall characteristics.

The NRCS TR-55 methodology for estimating T_c consists of summing the travel time (TT) of all flow types that may occur within a watershed: sheet flow (SF), shallow concentrated flow (SCF), and open channel flow (OCF). SF occurs over plane surfaces (e.g. a parking lot, grass field), typically has a maximum flow depth of 0.1 feet, travels in a wavelike manner, and has a maximum flow length of 100 feet. The travel time of SF is calculated utilizing the empirical relationship shown in Eqn 5.

$$TT_{SF} = \frac{0.007 \times (nl)^{0.8}}{(P_2)^{0.5} \times S^{0.4}} \quad \text{(Eqn 5)}$$

In Eqn 5, TT_{SF} is the travel time of sheet flow (hours), n is the Manning's roughness coefficient (unitless), l is the length of the longest sheet flow path (feet, maximum of 100 ft), P_2 is the 2-year, 24-hour precipitation depth (inches), and S is the average slope of the longest sheet flow path (ft/ft). P_2 can be acquired from NOAA Atlas 14 for a specific location. The n variable can be looked up in tables in the NRCS TR-55 manual or NEH-15 (Cronshey, 1986; NRCS & USDA, 2008).

SCF is flow that occurs in a defined channel with a flow depth between 0.1 and 0.5 feet and is estimated by calculating the average velocity through the specific channel. Velocity equations for specified channels are provided in NEH-15 (NRCS & USDA, 2008), and the travel time of shallow concentrated flow (TT_{SCF}) can be calculated by dividing the length of the channel by the average velocity and converting units as required.

OCF occurs in defined channels where surveyed cross-sectional data has been obtained, channels are visible on aerial imagery, or flow/stream- lines appear on hydrologic websites such as the United States Geological Survey (USGS), National Hydrography Database (NHD), or StreamStats. Manning's Equation (Eqn 6) is used to estimate the average velocity (V, feet per second) throughout the length of OCF, and similarly to SCF, V is used to estimate the travel time of OCF (TT_{OCF}) by dividing the length of OCF by velocity and converting units.

$$V = \frac{1.49}{n} \times R^{2/3} \times S^{1/2} \quad \text{(Eqn 6)}$$

In Manning's Equation, R is the hydraulic radius (feet), which is found by dividing the cross-sectional area of flow by the wetted perimeter. Per the Continuity Equation (Eqn 7), Manning's Equation is often rewritten in the form of Eqn 8:

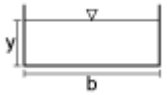
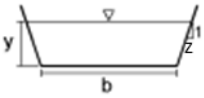
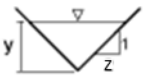
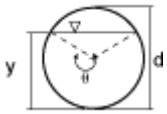
$$Q = V \times A \quad \text{(Eqn 7)}$$

$$Q = \frac{1.49}{n} \times A \times R^{2/3} \times S^{1/2} \quad \text{(Eqn 8)}$$

where Q is flow rate (cubic feet per second) and A is wetter area (feet squared). Geometric variables and computations used in Manning's Equation and OCF can be complex to calculate and differ by channel shape. Refer to Table 2-A for flow characteristics of typical channel

cross sections or the "The open channel flow calculator©." This tool was developed at the University of Auburn for automatic OCF computations in rectangular, trapezoidal, v-shaped (triangular) or circular channels (Fang, 2007). For computations in arched or elliptical pipes/channels, the designer is referred to Perez et al. (2015). For computations in parabolic pipes/channels, the designer is referred to the "Geometric Elements of a Parabolic Channel Cross Section" tool (Mejia & Ponce).

Table 2-A: Flow characteristics of typical channel cross sections.

Channel Shape	Area (A)	Wetted Perimeter (P)	Hydraulic Radius (R)	Top Width (T)	Hydraulic Depth (D)
	by	b+2y	$\frac{by}{b+2y}$	b	y
	(b+zy)y	$b+2y\sqrt{1+z^2}$	$\frac{(b+zy)\,y}{b+2y\sqrt{1+z^2}}$	b+2zy	$\frac{(b+zy)y}{b+2zy}$
	zy^2	$2y\sqrt{1+z^2}$	$\frac{zy}{2y\sqrt{1+z^2}}$	2zy	$\frac{1}{2}y$
	$\frac{1}{8}(\theta-\sin(\theta))d^2$	$\frac{1}{2}\theta d$	$\frac{1}{4}(1-\frac{\sin(\theta)}{\theta})d$	$2\sqrt{y(d-y)}$	$\frac{1}{8}(\frac{\theta-\sin(\theta)}{\sin(\frac{\theta}{2})})\,d$

*Circular pipe computations are to be calculated in radians.

2.1.3 Peak Flow Rate

For its simplicity, the Rational method is the most commonly used methodology to predict peak flow rates in the United States. It is an empirical relationship between peak discharge (q_p), rainfall intensity (i), a runoff coefficient (C), and drainage area (A) (Eqn 9).

$$q_p = C_{comp} \times i \times A \qquad \text{(Eqn 9)}$$

The rainfall intensity can be calculated for a given recurrence interval (e.g., 2-years or 5-years) for a duration equal to the watershed time of concentration, T_c. Lookup tables in Chapter 4 of TDOT's Drainage Manual can be used to determine C values based on surface types and conditions within the drainage area (TDOT). When multiple surface types or conditions are present within the drainage boundary area-weighting the C values is appropriate to obtain C_{comp}, a similar process as depicted in Eqn 4. In Eqn 7, the variables q_p, i, and A have units of cubic feet per second, inches per hour, and acres, respectively, while C is unitless. The Rational method was developed for use when drainage areas are less than 600 acres

(Kuichling, 1889); however, caution and engineering judgement are advised once watersheds exceed 10 acres.

The NRCS-TR 55 Graphical Peak Discharge method relates a unit peak discharge (q_u) multiplied with watershed area (A_m), runoff depth (Q_{CN}) and a ponding factor (F_p) to peak discharge (q_p) (Eqn 10).

$$q_p = q_u \times A_m \times Q_{CN} \times F_p \quad \text{(Eqn 10)}$$

In this equation, q_p is expressed in cubic feet per second (cfs), while q_u, A_m, and Q_{CN} (Eqn 1) are expressed in cubic feet per second per square mile per inch (csm/in), square miles (mi^2), and inches, respectively. F_p is governed by the percent area within the watershed that is ponds or swamps and can be looked up in tables in the NRCS TR-55 manual (Cronshey, 1986). The q_u variable is dependent upon T_c, appropriate rainfall distribution, P (ref. NOAA atlas 14), and I_a (Eqn 3). Utilizing the original NRCS TR-55 methodology, the rainfall distribution is Type II for the entire state of Tennessee and q_u can be determined from graphics (Cronshey, 1986) or hydrologic software. More recently, NOAA has created more refined and updated rainfall distributions, in lieu of the NRCS Type II distribution. Within the Ohio Valley geographic region, NOAA has defined four rainfall distributions: Type A, B, C, and D, all of which are represented in the state of Tennessee. These rainfall distributions are pre-loaded in certain hydrologic models (e.g., HydroCAD®) making the calculation of the q_u variable simple. Other software (e.g., WinTR-55) do not have the updated NOAA rainfall distributions incorporated into the models at this time. However, in WinTR-55, users can input custom rainfall distributions. Appendix A of the Georgia Soil and Water Conservation Commission (GSWCC) Manual for Erosion and Sediment Control provides resources to implement NOAA rainfall distributions in the NRCS models (GSWCC, 2016).

2.2 Estimating Sediment Yield

Sediment yield is a critical design element for EPSC measures on construction sites in regard to sediment storage capacity and maintenance. Erosion potential is dependent on four primary factors: soil erodibility, vegetative cover, topography, and climate. Understanding the parameters affecting soil loss helps with construction stormwater management by informing planners which project stages have the largest predicted soil losses. One of the most well-known models for estimating long-term soil loss is the Universal Soil Loss Equation (USLE), which was developed in the 1950s by the United States Department of Agriculture Agricultural Research Service (USDA-ARS), Soil Conservation Service (SCS, now known as NRCS), and Purdue University. The USLE model is on its second revision, the Revised Universal Soil Loss Equation 2 (RUSLE2), which was developed in the 1990s. Though the use of RUSLE2 is recommended for calculating soil loss on any development project, it is especially useful when site specific designs are required. The CGP (TDEC) requires that site

specific calculations be completed to design temporary sediment basins with a contributing drainage area of 25 acres or greater.

2.2.1 RUSLE2

2.2.1.1 Model

The RUSLE2 model is freely available at the USDA-ARS website and is recommended for use as opposed to performing hand calculations (https://fargo.nserl.purdue.edu-/rusle2_dataweb/RUSLE2_Index.htm). Foster and Toy (2005) and Toy and Foster (2007) are excellent resources for learning the software. The RUSLE2 model calculates sediment loss on slopes from rill and interrill erosion on an annual basis. The interrill erosion process (also known as sheet erosion), starts with raindrop impact detaching soil particles (i.e., sediment), thereby allowing the particles to move across the soil surface. Interrill erosion forms rills on the hill slope. Sediment is transported through the rills down the slope until the runoff slows enough to allow the deposition on either the land surface or in concentrated flow areas such as channels. Specific variables for RUSLE2 include annual erosivity (R), soil erodibility (K), slope length and steepness (LS), cover management factor (C), and a support practices factor (P) all of which are used to predict the average soil loss (A) from rill and interrill erosion (Eqn 11).

$$A = R \times K \times LS \times C \times P \qquad \text{(Eqn 11)}$$

Each variable in the RUSLE2 equation is determined based on a set of empirical equations and is dependent on a number of factors. Computation of variables requires caution and time; therefore, use of the model is advised. A brief explanation of the variables is provided, but for further specifics, the user is referred to the RUSLE2 manual (USDA, 2001).

The R-factor quantifies the amount and rate of runoff likely associated with rain by assessing raindrop impact on soil. This is impacted by weather, climate, and seasonal variations in temperature, rainfall, and wind. The R-factor has units of hundreds of [ft × tonf × in × (ac × hr × yr)$^{-1}$] and has typically been expressed as an annual average. However, in RUSLE2 the R-factor can be multiplied by a fractional amount of erosion to obtain daily, weekly, monthly, etc. values.

The K-factor is the rate of soil loss per rainfall erosion index plot, has a unit of [ton × ac × hr × (hundreds of ac × ft × tonf × in)$^{-1}$], and is influenced by temperature, precipitation, and intrinsic soil properties. In general, soils with high clay or sand content have lower values of K. In high clay soils, this low value is because the attraction forces between clay particles are often stronger than the forces applied to soils by raindrops. In sandy soils, it is due to the high infiltration capacity. Conversely, silty soils typically yield moderate to high values of K because particles can be easily detached by the forces imparted by raindrop impact and the soils are unable to infiltrate rainfall at a high enough rate.

Topography is accounted in RUSLE2 by considering the horizontal distance from the origin of overland flow to the point where either (1) the slope gradient decreases enough for deposition to begin or (2) runoff enters a defined channel and becomes concentrated (slope length). Topography is also accounted for by the gradient of the slope length (slope steepness). Slope length is typically limited to 400 feet, although longer sections may be applicable in certain scenarios. Together, slope length (feet) and steepness (%) are used to obtain a unitless LS factor from tables in Renard (1997) or within the RUSLE2 model.

The C-factor is a unitless variable reflecting the effect of cover crops and management practices (e.g., vegetation type, growth, application of biomass, crop rotation, conservation tillage, and surface roughness) on erosion rates. The C-factor is split into subfactors including canopy, ground cover, surface roughness, ridges, below ground biomass, soil consolidation, and antecedent soil moisture in which a set of predetermined equations are used to calculate their interactions related to soil loss.

The P-factor describes the ratio of soil loss with a specific support practice to the corresponding loss with upslope and downslope tillage. It is a unitless variable calculated from a product of P subfactors for individual support practices that can be standalone practices or used in series.

The product of all RUSLE2 variables is average soil loss expressed in tons per acre per year. However, decomposition of the R and K variables allows the user to better refine soil loss estimates for a user-specified timeframe. The average loss can also be viewed as the net sum of soil losses related to the following four processes:

- soil loss on the eroding portion the slope;
- soil detachment on the entire slope;
- conservation planning soil loss; and
- sediment delivery for the slope length.

2.2.1.2 Model Output and Summary

RUSLE2 provides multiple model outputs that can be used for gauging performance and for planning different management scenarios. One output value is the average soil loss for the user-specified timeframe. Soil loss values can be compared for different management scenarios and different timeframes to determine if EPSC measures or construction timing can help reduce estimated soil losses. RUSLE2 also outputs a critical slope length that indicates the maximum slope length before a contouring management practice (such as silt fence) begins to fail. If the actual slope length is greater than the critical slope length, the slope may be more vulnerable to failures of EPSC measures. Cover management systems that increase slope roughness or ground cover can increase the critical slope length. Other measures, such as grass strips or terraces, may also increase the critical slope length. It is

desirable for the actual slope length to be less than the critical slope length for erosion control applications. Model users should carefully review RUSLE2 model input and output information to check that the results are reasonable and consistent with accepted values from the manual.

2.2.1.3 Applicability

The RUSLE2 model can be used for planning, implementing, and designing EPSC measures as well as for construction planning and sequencing. Scenarios where RUSLE2 are particularly useful are listed below:

- Calculating a baseline estimated soil loss for comparison with other scenarios;
- Comparing slope erosion rates at various stages of construction;
- Ranking EPSC measures erosion control performance;
- Calculating estimated sediment yields for phased and timed projects to minimize soil exposure during rainfall events;
- Diverting runoff away from potential high erosion areas;
- Reducing overland flow path length where EPSC measures do not provide desired functionality;
- Displaying positive effects of stabilizing disturbed areas with vegetation, ground cover, biomass, etc.;
- Selecting local vegetation types for long term erosion control;
- Adding flat segments at the end of overland flow paths to promote sediment deposition; and
- Implementing sediment trapping devices.

RUSLE2 output may be used to evaluate the relative effectiveness of different EPSC measures; however, RUSLE2 does NOT apply in the following scenarios:

- Concentrated flow areas (e.g., gullies, ditches, streams);
- Undisturbed forestland;
- Erosion by piping;
- Erosion caused by snowmelt;
- Erosion by mechanical processes (mass movement such as landslides, movement by tillage operations);
- Organic soils;
- Slope lengths longer than 1000 feet;
- Slopes steepness greater than 100%;
- Sediment basins beyond small, simple designs; or
- Diversion engineering designs.

2.2.2 Other Models

As previously mentioned, RUSLE2 is suggested when determining annual site specific sediment storage volume. Site specific sediment loadings are required when the contributing drainage area to an outfall is greater than 25 acres. While various methodologies may be applicable, only one alternative (the sediment yield ratio) is outlined below. However, another model, the Modified Universal Soil Loss Equation (MUSLE), is mentioned and supplemented with a reference (Williams, 1975). MUSLE is an adjustment from USLE by incorporating runoff factors, making it applicable to individual rainfall events, rather than giving annual or long-term averages.

When intrinsic soil properties of the area to be disturbed are known (from local studies and tests), a simple sediment yield ratio relating the disturbed area with sediment yield can be implemented. In this instance, the bulk density of the sediment must be considered to obtain a sediment yield in tons. When local studies have not been conducted, a general rule is that one inch of disturbed/exposed soil will erode over a given acre per year, yielding 3,618 cubic feet of soil loss per acre of disturbed area per year. Therefore, when using this ratio for temporary sediment basin (Section 4.4.7) or sediment trap (Section 4.4.8) calculations, the sediment storage should be sized to hold the sediment yield of 3,618 cubic feet per acre of disturbed area. In addition, the basin and outlet structure must be sized to detain the runoff from the entire drainage area and treat the design storm that is required in the CGP (the 2-year, 24-hour or 5-year, 24-hour storm event). Use of the sediment yield ratio is limited to when there is sufficient knowledge on the soils, drainage basin, and climate. This generalized rule should not be used for areas that are over 25 acres; instead, a site specific design must be completed for the sediment storage required for sediment control measures (such as temporary sediment basins).

2.2.3 Design Concepts for Structural Sediment Controls

The first and most important step in erosion prevention and sediment control is minimizing soil disturbance and preserving the existing vegetative cover. Well planned projects avoid and minimize adverse impacts, and where disturbance is necessary, the least impactful construction processes are chosen. Final stabilization should be implemented as soon as possible. The design of sediment control practices relies on the predicted volume (or weight) of eroded soils (i.e., the resultant calculation from RUSLE2, MUSLE, sediment yield ratio, etc.), among other factors. These practices are designed to trap, and store suspended sediment carried by surface water from a construction site until maintenance operations remove the trapped sediment. A common sediment control practice utilized in Tennessee is a temporary sediment basin, which may be a required EPSC measure per the CGP on some construction sites. Other sediment control practices [e.g., sediment trap, silt fence, check dams, etc.

(Section 4.4)] can be implemented in series, combination, or alone to provide sediment trapping and stormwater treatment. These alternates may be necessary when site conditions make temporary sediment basins infeasible or when their implementation is anticipated to outperform sediment basins (Section 5 and 6).

For temporary sediment basins, the efficiency and effectiveness of trapping and storing sediment not only depends on the geometry and size of the practice but also the design components (sediment and water storage, forebay, principal spillway, outlet pipe and protection, emergency spillway, and dam embankment) and soil characteristics. The design procedures and functions of these components are specified in Section 4.4.7. The geometry can also impact the treatment efficiency especially when it comes to the flow path of stormwater in a temporary sediment basin. Longer flow paths from the inlet to outlet allow more time for finer sediments to settle out of the stormwater before exiting the sediment basin. Generally, settling velocity estimates are 3.28×10^{-3} to 3.28×10^{-1} feet per second for sands, 3.28×10^{-5} to 3.28×10^{-3} feet per second for silts, and less than 3.28×10^{-5} feet per second for clays. However, these ranges can vary based on factors such as water turbulence, particle size and density, and temperature (Winterwerp & Kranenburg, 2002). Depending on the particle size to be removed in a sediment basin, the required retention time of that particle varies drastically (Table 2-B), thereby influencing the flow path length.

Table 2-B: Example settling times for sand, silt, and clay are shown for comparison.

Particle Type	Approximate Particle Size (μm)	Settling Velocity (ft/s)	Depth of Basin (ft)	Settling Time (s)	Settling Time (min)	Settling Time (hr)
Sand	202	3.28E-02	4	122	2	0.03
Silt	20	3.28E-04	4	12192	203	3
Clay	6	3.28E-05	4	121920	2032	34
Clay	2	3.28E-06	4	1219200	20320	339

The table above is only shown as an example to compare the settling time for different soil types. In no way does this table show the required settling time to meet regulatory requirements for a sediment basin design. The comparison shows that the finer particles can take upwards of 10,000 times longer to settle than coarse particles.

Effective EPSC measure design and sizing depend on accurate analyses of runoff volume, peak flow rates, and sediment yield, informed by established hydrologic methodologies and supported by computational tools for efficient and precise scenario evaluation. Ultimately, the responsibility for ensuring a compliant and effective design rests with the design engineer.

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Chapter 3 - Erosion Prevention and Sediment Control Plans

3.1 Purpose

EPSC measures are designed to reduce, capture, store, and treat sediment laden runoff from construction sites and are commonly categorized by their purpose. Some of the categories include erosion prevention, runoff velocity control, and sediment control, all of which will be discussed further in Chapter 4. EPSC measures are most effective when implemented from a set of design plans that include the placement, size, quantities, and detailed drawings for each of the proposed EPSC measures.

The first and most important step in erosion prevention and sediment control is minimizing soil disturbance and preserving the existing vegetative cover. Well-planned projects avoid and minimize adverse impacts, and where disturbance is necessary, the least impactful construction processes are chosen. Final stabilization should be implemented as soon as possible.

EPSC plans must be site specific due to the unique combination of features present at each construction site. Site specific features may include confined or narrow areas, depth to ground water table, depth to bedrock, slopes, soil textures and classifications, proximity to receiving waters, etc. When creating EPSC plans, site specific knowledge is paramount in preventing sediment laden discharges from the construction site, a universally important process for protecting receiving waters.

3.1.1 System Approach Design

Due to the unique constraints of every construction site, typical EPSC measures may not be feasible to implement; therefore, multiple, less constrictive measures can be paired in series to promote stormwater treatment. That is, a system of EPSC measures may be as effective or more effective in combatting the export of sediment than a single, more resource-intensive measure. Subsequently, designers and developers may consider implementing a system of less constricting measures to obtain the same expected result as the more intensive EPSC measure(s). This concept is known as treatment trains of EPSC measures. This systems approach can be used as treatment equivalency to replace a sediment basin or sediment trap. For example, on most construction sites, a sediment basin would likely remove the largest quantity (or mass) of sediment, but it may be infeasible to implement on some construction sites due to site constraints. Thus, designers can implement a treatment train of multiple, less extensive measures (e.g., rock check dams, silt fence, and sediment traps), to achieve the same treatment as a sediment basin. Designers and developers are referred to the chapter entitled “Equivalency and Treatment Trains” of this manual (Section 5) to better understand this process.

3.2 Erosion Prevention and Sediment Control Stages

EPSC measures must be implemented in a timing that parallels construction. For example, it would be illogical to implement pollution prevention measures when construction is near completion just as it would not be practical to implement erosion prevention measures (such as erosion control blankets) before soil is exposed during the initial phases of construction. Therefore, EPSC measures are to be shown in three stages: initial, interim, and final to provide effective management of onsite sediment.

All EPSC measures are to be located within the limits of disturbance. Offset the limits of disturbance from the proposed grading limits to ensure sufficient working room for construction of slopes and placement of EPSC measures.

For sites under five acres of disturbance, a single plan sheet may adequately address all three EPSC stages. For example, a water line installation may have a single EPSC plan sheet that provides design information for the initial, interim, and final stages. If the entire alignment of the proposed pipeline does not fit on one sheet, then multiple sheets must be provided for the EPSC plans, but multiple sheets may not be needed for each stage. Multiple sheets would be required to show the EPSC plans for the entire project area not for each separate stage since all three stages can be addressed on the sheet for that segment. In this example, the initial and interim stages may have the same proposed perimeter controls and may be indicated with a note: “Initial and interim EPSC measures are the same. Replace and repair EPSC measures as needed.” An additional note to address the final stage could state “For the final stage, grade back to preconstruction contours and stabilize with seed and mulch.” These notes would provide sufficient EPSC plan information on a single sheet for a defined area on the construction site of a water line installation.

3.2.1 Initial Stage

The initial EPSC stage shall establish perimeter controls to protect existing drainage features, outfalls, water resources, and offsite property. Initial stage EPSC measures must be in place prior to any earth-disturbing activities to prevent sediment from leaving the construction site. All measures must be inspected and maintained throughout the entire construction period.

3.2.2 Interim Stage

The interim EPSC stage shall display the proposed measures that are to be in place during grading operations. During this stage, drainage patterns, outfall locations, and erosion potential can change with site grading. This may require interim substages to address changes in site hydrology and sediment loss. As more soil is exposed throughout construction, redundancy of measures can further decrease sediment loss. Ultimately, the

objectives for this stage are to show additional measures can be used to prevent (or how modifications to existing measures can be used to continue to prevent) sediment from migrating offsite, despite earth disturbing activities altering site conditions.

3.2.3 Final Stage

Activities in the final stage include final grading, topsoil installation, and slope and ditch stabilization. Per the CGP, stabilization must occur within 14 days of temporarily or permanently ceased construction activities, or within seven days if slopes are greater than 35% (TDEC). Remove temporary EPSC measures from the plan sheets; only show the permanent measures.

3.3 Site Planning and Preparation

Site planning and preparation requires a site visit to assess pre-construction conditions. Notable features to assess include soil types and classifications, drainage patterns, existing vegetation, existing structures, existing pipe sizes, and topography. Planning documentation should indicate if soils are highly erodible or contaminated as well as the location of discharge points, steep slopes and slope lengths, and natural features requiring protection. Documenting these features is known as fingerprinting a site and the process is critical to ensure no damage to natural waterways or wetlands result from construction.

3.3.1 Outfalls

In the CGP, the terms “outfall” and “point source” are synonymous and are defined as any discrete conveyance that discharges runoff as concentrated flow into waters of the state, offsite locations, or storm sewer systems (TDEC). All outfalls must be clearly marked on plan sheets, thereby also identifying inspection locations at which to gauge the effectiveness of EPSC measures. All outfalls have an associated drainage area. Drainage areas may include offsite drainage as well as a portion or all of the disturbed area. Per the CGP, offsite drainage must be included in outfall drainage areas since runoff following a flow path through the construction area can transport sediment and further erode soils from the disturbed area. Therefore, all runoff created within or routed through the limits of disturbance requires treatment. Planners must ensure selected EPSC measures have the capacity to retain, store, and treat the associated runoff from each drainage area before discharged via an outfall.

Outfalls can be challenging to identify. Outfalls are not exclusively defined as a point of hydrologic analysis, but also where construction site inspections must occur. Several complexities and common issues arise when identifying outfalls on a construction site:

- Identify where run-on enters the limits of disturbance as inflow, not an outfall. The inflow drainage area must be included in the total drainage area for the properly

identified outfall, unless a structure is in place to divert the unpolluted water around the limits of disturbance;
- If a construction site is adjacent to a stream, do not identify outfalls within the stream but place them on the bank of the stream; and
- When disturbance is limited to a subsection of a property, identify the outfall where concentrated flow is discharged from the limits of disturbance, not the downstream property line. The one exception to this is if the downstream property line and downstream subsection's limit of the disturbance overlap.

When outfalls are placed in a stream or outside of the disturbed area, the associated drainage area includes runoff from upstream tributaries and undisturbed sections of the property. By misidentifying the outfall, design of all EPSC measures would be required to include this additional runoff volume in their storage capacity. Outfalls are important to identify correctly, as they are the most downstream inspection point for each drainage area. This is where the inspector can gauge the effectiveness of all EPSC measures within each drainage area at preventing sediment loss.

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3.3.2 Best Management Practices for Work Near Streams

Construction on and around streambanks results in removed and degraded vegetation and soil disturbance, thereby impairing streambank stability. Soil particles are dislodged from the bare soil when heavy machinery and equipment disrupt the soil structure. Streambank destabilization and highly erosive flows during storms subject the stream to excessive sediment inputs and habitat loss. Protecting and stabilizing streambanks from active construction is critical to preserve the land and ecosystem structure. Further, it is important to plan locations where heavy machinery can access or cross the stream if needed for construction purposes. When physical alterations to a stream or its banks are necessary, an ARAP (Chapter 0400-40-04) is required. Additionally, work that would involve discharge of dredged or fill material below the ordinary high-water mark of jurisdictional streams, could require written authorization from USACE under Section 404 of the Clean Water Act (refer to Section 1.3.2 of this manual for further details). Several EPSC measures are included in this handbook as guidance when working in or around streams. Coverage under the CGP does not infer coverage under an ARAP or any other potentially necessary permit.

The most common stabilization measure is riprap, which is large angular rock that resists erosion due to its size, weight, and interlocking nature when set in place. Riprap gradations are differentiated by the size of stone. In Tennessee, Class A-1, B, and C are most common and have median stone dimensions (D_{50}) of nine, 15, and 20 inches, respectively. However, stone dimensions within each gradation can range from the upper size in the lower gradation to the lower sizer in the higher gradation. These ranges in size help fill void spaces between interlocking stones. Less common gradations may be considered for use when necessary and with proper approval from the design engineer. In addition to riprap gradation, riprap can also be classified by type: conventional, soil filled, and void-filled.

Conventional riprap is plain angular rock. Due to growing environmental concerns, minimize the use of this type where possible. Conventional riprap is most ideal where vegetation is not planned such as under bridges and pipe outfalls (MHFD 2024a; TDOT). Due to larger void spaces, conventional riprap requires placement of bedding material or geotextile fabrics between the stone and natural subgrade to prevent piping. Soil riprap is two-thirds conventional riprap filled with one-third native (top)soil, a combination which reduces void spaces. Soil riprap may include an added, uppermost layer of topsoil, providing a medium for vegetation. The combination of vegetation and riprap eliminates the need for an underlying geotextile. Soil riprap is ideal for bioengineered streambank protection, swales, and buried emergency spillways (MHFD, 2024a). Void-filled riprap is a mix of conventional riprap, cobbles, gravels, sands, and soils which creates a dense, void-filled composition that keeps flowing water above the surface and prevents erosion. Void-filled riprap mimics natural riffle-rock material and is ideal in continuous streamflow applications (MHFD, 2024b).

3.3.2.1 Bank Protection



Source: TDEC

Definition and Purpose

Bank protection is the stabilization of the side slopes of a waterbody. This can be vegetative, structural, or a combined method, such as bioengineered soil stabilization (See Section 3.3.2.2). Bank protection is necessary to repair and/or prevent erosion in areas where increased runoff rates and volumes are anticipated from development, construction, or stream alteration.

Appropriate Applications

Bank protection is appropriate where there is existing erosion or erosion is anticipated. This could be along straightened channels, in locations of (upstream) development, exposed streambanks, or where highly erosive flows have been observed.

Limitations and Maintenance

Few limitations exist for this practice as long as the protection is applied appropriately so as to withstand erosive forces and is firmly secured so the practice cannot become dislodged and migrate downstream. Otherwise, bank protection may require continual replacement or maintenance. Timing limitations exist for vegetative measures as they will not immediately stabilize the banks. Bank protection measures must not reduce the hydraulic capacity of the stream.

Installing bank protection in jurisdictional waters requires additional permits, such as an ARAP, and therefore, both the conditions of the CGP and ARAP must be followed. A Section 404 permit from USACE may also be required, and if the proposed bank protection is to be completed in a TVA reservoir, a TVA 26a permit may also be necessary. Local permitting may be required in addition to state and federal permitting requirements. Consider the criteria and conditions of the necessary permits during the planning stages of the project and EPSC plans.

Planning and Design Considerations

Several options exist for bank protection, including grass, trees, conventional, soil, and void-filled riprap, concrete, fabric-formed revetments, grouted riprap, erosion control blankets, turf reinforced matting, coir matting, benching, geotextile containment, etc. Some of these bank protection measures can be used in conjunction with bioengineered soil techniques as presented in Section 3.3.2.2 (GSWCC, 2016). It is ideal to install bank protection when a stream diversion (Sections 3.3.2.3 and 3.3.2.4) is in place (i.e., dry conditions). This prevents the transport of disturbed sediment downstream. Consider the following site specific conditions when selecting bank protection materials (ALSWCC, 2018):

- The overall condition of the stream within and adjacent to the reach is to be stabilized;
- Current and future watershed conditions;
- Velocity at full channel flow;
- Shear stress at full channel flow;
- Sediment load in the stream;
- Channel slope;
- Controls for bottom scour;
- Soil conditions;
- Present and anticipated channel roughness;
- Compatibility of selected protection with other improvements at the site;
- Changes in channel alignment; and
- Fish and wildlife habitats.

A well-conceived bank protection design consists of an interdisciplinary team, including (but not limited to) engineers, hydrologists, and wildlife biologists. Key considerations include stream geomorphology, stabilization of the channel bottom, and appropriate selection of bank protection measures based on velocity and shear stress. The channel bottom must be stabilized before installing bank protection to enhance long-term success. Grade control may be necessary to prevent stream downcutting (ALSWCC, 2018). Vegetated protection is typically preferred as it is inexpensive and resembles natural stream characteristics. However, vegetated banks do not provide immediate stabilization and do not provide as much stability under high flow conditions as other, typically structural materials (Tables

3.3.2.1-A and 3.3.2.1-B). Design velocities are to be determined by the designers under the appropriate design storm, which may be a local regulation. If no design storm is specified due to the project location and local government ordinances, utilizing the upstream channel capacity, estimated bankfull velocity, or the 2-year, 24-hour design storm are appropriate alternatives.

Table 3.3.2.1-A: Velocity thresholds for bank protection materials. Sources: FHWA (1998) and TDOT.

Material	Maximum Velocity (ft/s)
Bare Soil	-
Silt of fine sand	1.5
Sandy loam	1.75
Silt loam	2
Stiff clay	3.75
Ordinary firm loam	2.5
Fine gravel	2.5
Graded, loam to cobbles (noncolloidal)	3.7
Graded, silt to cobbles (colloidal)	4
Alluvial silts (noncolloidal)	2
Alluvial silts (colloidal)	3.7
Coarse gravel (noncolloidal)	4
Cobbles and shingles	5
Shales and hard pans	6
Sod	4
Lapped sod	5.5
Vegetation	Table 3.3.2.1-B
Conventional Riprap (Class A1, I)	5
Conventional Riprap (Class B, II)	10
Conventional Riprap (Class C, III)	12
Conventional Riprap (Class C, IV)	15
Conventional Riprap (Class D, V)	20

Table 3.3.2.1-B: Maximum velocity threshold for vegetated bank protection on slopes. Source: Adapted from TDOT

Vegetation Type	Exit Channel Slope Range (%)	Maximum Velocity[a] (ft/s)
Bermudagrass	0 - 5	6
	5 - 10	5
	> 10	4
Kentucky Bluegrass, Buffalo Grass, Smooth Brome	0 - 5	5
	5 - 10	4
	> 10	3
Grass Mixture	0 - 5	4
	5 - 10	3
Alfalfa, Crabgrass	0 - 5	2.5
Common Lespedeza, Sudangrass	0 - 5	2.5

[a]Based on erosive soils

Even if velocity permits a certain protective material, shear stress, the force per unit area exerted parallel to the streambank, must be checked to ensure compatibility. Streambanks are considered stable when the velocity criteria are met, and permissible shear stress (Table 3.3.2.1-C) is less than the tractive force (TDOT). Tractive forces are the hydrodynamic forces of flowing water in a channel. In a uniform flow, the tractive force is equal to the effective component of the gravitational force acting on the body of water, parallel to the bottom of the channel. Thus, the maximum shear stress for a straight channel occurs at the channel bottom and is less than or equal to the shear stress at maximum depth, which can be expressed as:

$$\tau_{max} = \gamma \times d \times S \qquad \text{(Eqn 12)}$$

where τ_{max} is the maximum shear stress in pounds per square foot, γ is the unit weight of water (62.4 pounds per cubic foot), d is the maximum depth of flow in feet, and S is the average bed slope or energy slope in feet per foot.

Concentrated flow traversing a bend imposes a higher shear stress on the inner bank within the curve and the outer bank downstream of the curve compared to a straightened reach. Maximum shear stresses in bends are a function of the ratio of the radius of curvature, R_C, to the channel bottom width, b. As this ratio decreases, the bend becomes sharper and shear

stresses increase. Maximum shear stress in a bend can be calculated by the following equation:

$$\tau_{bend} = \tau_{max} \times K_b \quad \text{(Eqn 13)}$$

where τ_{max} is calculated from Eqn 12 and K_b is a dimensionless factor accounting for the increased stress. This dimensionless factor is calculated by Eqn 14.

$$K_b = 2.36 \times e^{-0.082 \times R_C / b} \quad \text{(Eqn 14)}$$

Table 3.3.2.1-C: Shear stress thresholds for bank protection materials. Sources: FHWA (1998) and TDOT.

Lining Category	Lining Type	Permissible Unit Shear Stress	
		(lb/ft^2)	(Pa)
Erosion Control Blanket[a]	Type 1	1.5	72
	Type 2	1.75	84
	Type 3	2	96
	Type 4	2.25	108
Turf Reinforced Mat[a]	Type 5 - unvegetated	3	144
	Type 5a	6	288
	Type 5b	8	384
	Class 5c	10	480
Grass	Class A	3.7	177
	Class B	2.1	101
	Class C	1	48
	Class D	0.6	29
	Class E	0.35	17
Conventional Riprap	Class A1, I	3	144
	Class B, II	4	192
	Class C, III	5	239
	Class C, IV	6	287
	Class D, V	7+	335
Bare Soil	Non-cohesive	See Hydraulic Engineering Circular No. 15	
	Cohesive		

[a]General values based on vendor information, refer to Section 4.2.6.10 for detailed information.

When working around a water resource, such as a stream, consider the different zones within the riparian area and their susceptibility to erosion (NCDEQ, 2013). Typically, riparian areas consist of five zones, including toe zone, bank zone, overbank zone, transitional zone, and upland zone (Figure 3.3.2.1-A), though each zone may not be prevalent in streams impacted by development.

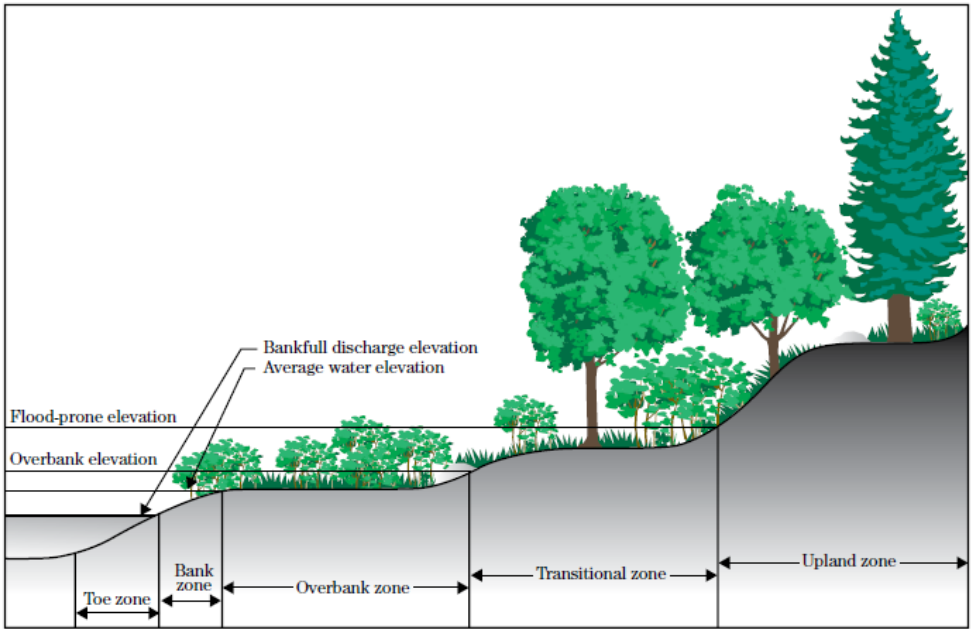


Figure 3.3.2.1-A: Riparian zones. Source: NRCS (2007).

The toe zone is located below baseflow water levels. This zone is perhaps the most critical to be stabilized, as it is continuously subjected to flow (NRCS, 2007). Because this zone is inundated (with the exception of drought periods), it is likely that no woody vegetation exists, nor would it be feasible to introduce woody vegetation as a protective measure. In the toe zone, nonliving protection (most commonly, structural) is most ideal.

The bank zone is the area situated between the typical water level and the bankfull discharge height. In this zone, the potential for erosion is high, but only during stormflow. Furthermore, this zone experiences wet and dry cycles, debris deposition, and in winter months, it can be subjected to freeze-thaw cycles. These exposures may exacerbate erosion potential. This zone is often covered by early-growing herbaceous and small woody plants with flexible stems. Such species may include willows, dogwoods, elderberries, and small shrubs (NRCS, 2007). Because this zone is subjected to erosive forces and there is a low potential of protection from natural vegetation, bank protection is likely necessary. Planting vegetation may provide adequate protection in small water bodies, especially if the upstream has little development. However, nonliving protection will likely be more effective.

The overbank zone is located above the bankfull discharge elevation, which floods approximately every two to five years. It is typically a flat area because it has been formed by sediment deposition when flooded. Vegetation found in this zone is generally flood tolerant, provides shade for the stream, and offers a riparian habitat for wildlife. Protection usually does not need to extend into this zone as long as existing vegetation is present and healthy (NRCS, 2007). However, support this zone with vegetative protection if needed. Typically, this zone is where the preservation of riparian buffer zones (Section 3.3.3.2) would begin.

The transitional zone and upland zone are seldomly subjected to erosion from the stream. The transitional zone may be flooded approximately every 50 years while the upland zone would be subjected to erosion from sheet flow and land disturbance (NRCS, 2007). Bank protection measures are not applicable in these zones.

When the velocity and shear stress align with vegetative design, incorporating diverse plant species is ideal. Consider natives first. However, there are difficulties when it comes to establishing natives, such as acquiring 70% vegetative cover and time to germination. Native vegetation can be supplemented with nurse crops in order to provide erosion control while the natives germinate and grow. See Section 4.2.6.11 Vegetation and Landscaping: Permanent for further details on native establishment. If native plants are not feasible, then consider non-invasive species for stabilization. Where high velocities, channel bends, steep slopes, or highly erodible soils exist, structural protection will be more desirable than vegetation. Common structural bank protection materials include conventional riprap, gabions, fabric-formed revetments, and reinforced concrete walls.

Conventional riprap is one of the most commonly used materials for bank protection, providing stability and erosion resistance in high-flow environments. To ensure its effectiveness, size riprap to withstand the expected flow conditions. The first step in this process is to determine the design velocity and maximum shear stress, based on either bankfull flow or a more conservative design storm. Machined riprap is categorized into different classes based on median stone size (D_{50}). Class A-1 has a D_{50} of nine inches and is placed at a minimum depth of 18 inches. Class A-2, though similar in material, is hand-placed at a depth of 12 inches and is generally not recommended due to its reduced stability. Class B and C


Source: NCDEQ (2013)

riprap have a D_{50} of 15 and 20 inches, respectively. For slopes steeper than 3H:1V, refer to Chapter 4 of the FHWA publication HEC-15 to ensure stability. Permissible velocities for riprap classes are presented in Table 3.3.2.1-A. However, using shear stress calculations provides a more accurate assessment of stability, and these thresholds must also be checked (Table 3.3.2.1-C). Additional considerations to prevent failure include ensuring that bank slopes do not exceed a maximum of 2H:1V, incorporating geotextile filter fabric between the soil and conventional riprap to prevent piping, and extending the riprap toe at least one foot below the streambed. Ideally and when possible, extend riprap up to the 2-year water surface elevation or bankfull flow unless a combination of vegetative measures is determined to provide sufficient protection. Where possible, consider the use of soil riprap or void-filled riprap in place of conventional riprap as a bioengineered soil technique (Section 3.3.2.2).

Gabions are stone-filled wire baskets that are semi-flexible and permeable. Because of their flexibility, gabion structures remain structurally sound and function during earth movement. Void spaces in gabions absorb energy from flowing water, thereby reducing velocity. Because this material is permeable, place a geotextile fabric between the gabions and soil subgrade to prevent any piping. Stone size is less critical for gabions on steep slopes (as opposed to riprap alone) because the wire mesh allows the material to act as a single unit. However, the stability and allowable shear stress of wire-enclosed riprap depends on the integrity of the wire mesh. In channels with high sediment yields, the wire mesh may be abraded and could potentially fail. In such conditions, the use of these structures are not advised. Check manufacturer specifications for permissible shear stresses of wire.


Source: NCDEQ (2013)


Source: TDEC

Fabric-formed revetments are manufactured, large, quilted envelopes that are pumped with fine-aggregate grout onsite. These systems are inexpensive and take only a few minutes to set in place. Essentially, they are cast-in-place rigid linings that are durable, flexible, and semi-permeable. Due to their flexibility, fabric-formed revetments are ideal with stream banks having irregular surfaces. Furthermore, the permeability of geotextile fabrics allows water filtration, which can be beneficial to the local ecosystem. There are various options for revetments on the market. Be sure to check manufacturer specifications to ensure the proper selection for each site and permissible shear stresses. Additionally, follow all manufacturer specifications during installation.


Source: GSWCC (2016)

Reinforced concrete retaining walls may be necessary in urban settings or constrained sites to replace cut or fill slopes, in areas with poor soils, or where a wall is necessary to achieve stable slopes. Designing retaining walls is a complicated process and site specific; thus, an engineered design is required. Many factors need to be considered, such as allowable stresses and forces outside and within the wall, allowable height, wall thickness, foundation design, footers, bearing values of soils, a factor of safety, etc. The foundation must include a method of draining excess water from behind the wall. Begin and end all structural protection methods along stable reaches of the stream.

The velocity and shear stress discharging from the protected to a natural stream reach must be considered, particularly if the reach was straightened. Ensure protected banks do not increase the velocity of water such that downstream erosion occurs. If the velocity or shear stress is too high, consider designing a transition in the bank protection to slow the erosive forces, thereby protecting the downstream channel bed and banks.

Example Application

Always ensure that the maximum velocity and shear stress are less than the permissible velocities and shear stresses of bank protection. Maximum velocities and shear stresses in streams are to be calculated by design engineers registered in Tennessee.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.
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FHWA. (2005). *HEC 15: Design of Roadside Channels with Flexible Linings.*
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TDOT. *Drainage Manual Ch5.*

3.3.2.2 Bioengineered Soil Bank Stabilization



Source: TNWRRC

Definition and Purpose

Soil bioengineering is defined as the use of living and nonliving plant materials in combination with natural and synthetic support materials for slope stabilization, erosion reduction, and vegetative establishment. These techniques incorporate natural and readily available plant materials such as root wads, coconut fiber rolls, and rock or log vanes to direct stream flow away from eroding banks and stabilizing the bank.

Appropriate Applications

Bioengineered soil bank stabilization is appropriate anywhere repair or protection from erosive forces on the banks is needed. This could be along straightened channels, in urbanized watersheds, exposed streambanks, or where highly erosive flows have been observed or are anticipated. Native vegetation is the preferred method of bank stabilization. However, when vegetation cannot provide adequate stabilization, bioengineered methods are favored.

Limitations and Maintenance

Bioengineered soil stabilization measures must be affixed in place so they cannot become dislodged migrate downstream. Continual repair or maintenance may be incurred if the stabilization practice is under-designed and cannot withstand the shear forces in which it is

subjected. This can be costly and more impactful than initially designing a more stable option. Any implemented practice must not reduce the hydraulic capacity of the stream.

Installing bioengineered soil bank protection in jurisdictional waters requires additional permits, such as an ARAP, and therefore, both the conditions of the CGP and ARAP must be followed. A Section 404 permit from USACE may also be required. If the proposed bioengineered soil bank protection is to be completed in a TVA reservoir, a TVA 26a permit may also be necessary. Local permitting may be required in addition to state and federal permitting requirements. Consider the criteria and conditions of the necessary permits during the planning stages of the project and EPSC plans.

Planning and Design Considerations

When selecting the best-suited bioengineered technique, it is important to have a clear understanding of the ecological systems in adjacent areas. Plant selection and the techniques used will play an initial role in site stabilization and, ultimately, serve as the foundation for the ecological restoration of the site. Long-term establishment and health of herbaceous, woody vegetation is critically important to the physical and biological functions of streams. See Section 3.3.2.1 Bank Protection for the different planting zones within the riparian buffer zone.

Bioengineered soil bank stabilization is especially useful as a transition between conventional, inert bank stabilization and natural downstream banks. Abrupt transitions from projects, such as conventional riprap, to natural channel linings are often prone to scour. Established bioengineered soil treatments can act to protect and reinforce the transition and reduce the possibility of washouts and flanking (NRCS, 2007). Combinations of vegetative and structural protection can often be used in place of most structural measures. As with structural measures, start and end all combined methods along stable reaches of the stream.

Cellular confinement matrices are commercially available products comprised of heavy-duty polyethylene and formed into a honeycomb-shaped matrix. The product is flexible to conform to irregular surfaces and the honeycombs can be filled with soil, sand, gravel, or cement. When soil is filled into the honeycombs, it is usually ideal to plant vegetation in addition. Install all cellular confinement systems according to manufacturer specifications.


Source: Yee (2012)

Live cribwalls are a bioengineered technique that consists of a hollow, box-like interlocking arrangement of untreated logs or timber. They are appropriate to use at the base of a slope where a low wall may be required to stabilize the toe and reduce steepness. They are also appropriate where space is limited, and a vertical structure is required. The live walls are natural materials, thereby promoting environmental sustainability and allowing for the growth of plants within their structure, which can help with long-term bank stability. Cut live branches one-half to two inches in diameter. Construct the crib from logs or timbers from four to six inches in diameter or width. Branch lengths will vary depending on crib size. Starting at the lowest point of the slope, excavate loose material two to three feet below the ground elevation until a stable foundation is reached. Excavate the back of the stable foundation (closest to the slope) slightly deeper than the front to add stability. Place the first course of logs or timbers at the front and back of the excavated foundation, approximately four to five feet apart and parallel to the slope contour (Figure 3.3.2.2-A). Place the next set of logs or timbers orthogonal to the slope on top of the previously set timbers. Place each set of timbers in the same manner and affix them to the preceding set. Place live branch cuttings on the top of each cribwall structure with growing tips oriented outward. Backfill the cribwall, compact the soil for good root-to-soil contact, and apply seed and mulch.


Source: Tardio et al. (2018)

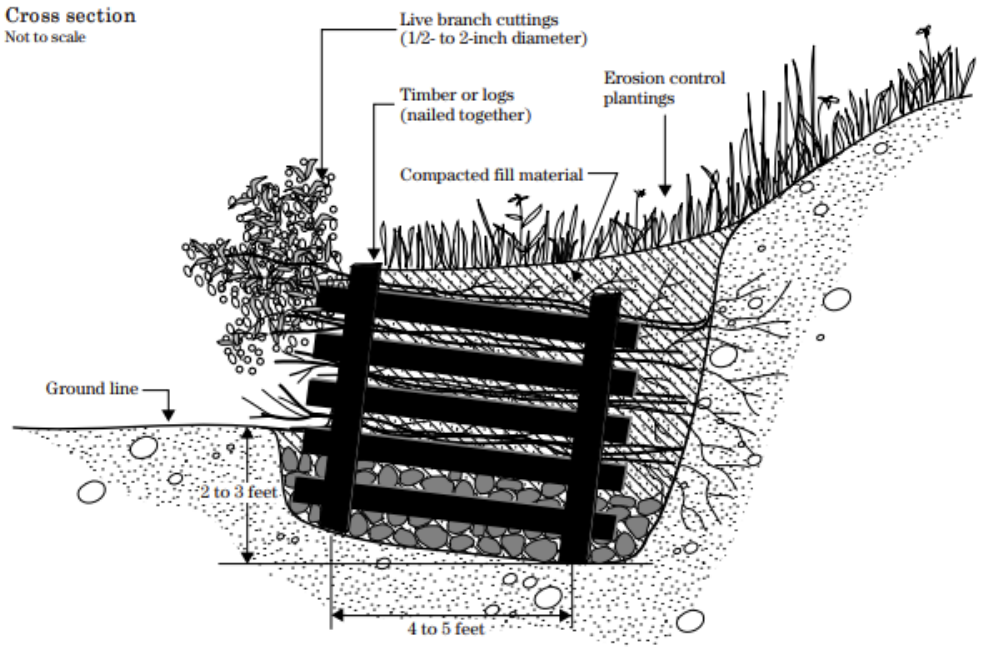


Figure 3.3.2.2-A: Live cribwall details. Source: NRCS (1996, 2021).

The Tennessee Cedar Revetment is a bioengineered soil bank technique that combines small, locally harvested eastern red cedar trees with 700-gram coir matting to form a flexible, interlocking structure that traps sediment and protects eroding banks. Constructed with cedar branches as the internal framework and encased in coir matting, each revetment unit is typically 10 feet long and 14 to 22 inches in diameter, forming a sturdy, yet flexible structure that conforms to the streambank. This method is most suitable for small to medium-sized streams with active erosion but where heavy equipment access is limited or undesirable, especially in forested areas where preserving vegetation is a priority. Installation begins downstream, using duckbill anchors and aircraft cables to tightly secure each revetment into the bank, layering them in overlapping sequences to prevent water from getting behind the structure. Revetments may be stacked in multiple rows depending on bank height and are often paired with rock toes or vanes for added protection. Once in place, the coir matting filters suspended sediment while the cedar adds structure, creating a new, reinforced bank that supports natural revegetation. Tennessee Cedar Revetments offer a cost-effective, minimally invasive, and volunteer-friendly alternative to conventional erosion control, with a proven track record of durability and ecological compatibility in Tennessee streams (Gregg & McFadden).

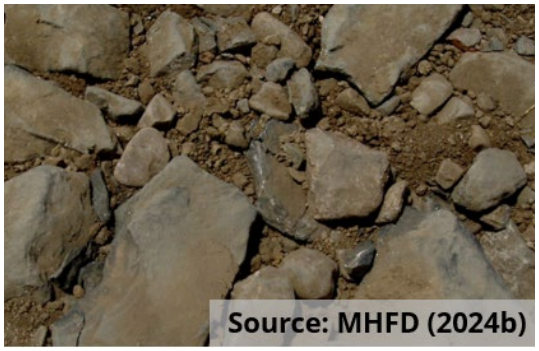

Source: Gregg & McFadden

Soil riprap is a bioengineered soil technique that mixes conventional riprap with natural soil or topsoil. The ideal mixture is two-thirds conventional riprap and one-third soil, which fills the voids between the angular stone. Mix the materials to a uniform consistency with a front-end loader or excavator. Prior to installation, ensure that the subgrade is excavated to the proper depth to account for the soil riprap upon placement and subsequent compaction. Because the soil fills the void spaces, erosion due to piping is not a concern, and therefore an underlying geotextile or bedding material is not necessary, as with conventional riprap. As the soil riprap is installed, ensure it is compacted such that the angular stone forms interlocking compartments to hold the soil in place. Additionally, ensure there are no excessively large pockets of soil, which may indicate weak spots. Once installed, all void spaces should be adequately filled


Source: MHFD (2024a)

and compacted by visual inspection. For some projects, an additional layer of topsoil may be added to the top of the soil riprap profile to provide a more suitable growing medium. This practice is particularly useful where aesthetics are important, as vegetation will effectively hide the riprap (MHFD, 2024a). Ensure the soil and conventional riprap are stored separately onsite, as pre-mixed compositions that are not compacted may allow the soils to migrate downward, resulting in a nonuniform matrix.

Void-filled riprap is a soil bioengineered technique that consists of a composition of conventional riprap, cobbles, gravels, sands, and onsite (or natural) soils. The materials in void-filled riprap will form a dense, interlocking composition that more naturally mimics natural stream flow compared to soil riprap or conventional riprap. Thus, this practice is ideal where continual and/or larger flows are anticipated. Using a front-end loader or excavator, mix an initial test batch of all void-filled riprap materials until a uniform consistency is achieved. Make note of the ratio of materials used such that the test batch can either be recreated or amended. Amending material proportions should be done during this initial test batch such that the finalized ratios most accurately resemble natural riffle material. Prepare the subgrade and place the void-filled riprap in the same manner as soil riprap. Ensure that the mixing and placement of the void-filled riprap results in some of the larger riprap flush and visible on the surface, with other materials organized to minimize the voids. Before compacting, inspect the void fillings. It may be necessary to add bedding or cobbles to the surface and "wash it in" with a high-pressure hose to fill any voids. After which, compacting can occur. These techniques ensure stability such that the void fillings are locked in place and cannot be eroded away or migrate downwards (MHFD, 2024b). Lastly, ensure void-filling materials are stored separately on-site similar to soil riprap.


Source: MHFD (2024b)

Vegetated gabions are a bioengineered soil technique that combines layers of live branches and gabions (Section 3.3.2.1). This practice is appropriate at the base of a slope where a low wall is required to stabilize the toe of the slope and reduce its steepness. It is not designed to resist large lateral earth stresses. Ideal use is where space is limited and a more vertical structure is required. Keep the overall height, including the footing, less than five feet. Cut live branches one-half to one inch in diameter and long enough to reach beyond the rock basket structure into


Source: TDOT

the backfill. Starting at the lowest point of the slope, excavate loose material two to three feet below the ground elevation until a stable foundation is reached (Figure 3.3.2.2-B). Excavate the back of the stable foundation (closest to the slope) slightly deeper than the front to add stability and ensure rooting. Place the wire baskets in the bottom of the excavation and fill with stone. Backfill between and behind the wire baskets. Place live branch cuttings on the wire baskets orthogonal to the slope, with the growing tips oriented away from the slope and extending slightly beyond the gabions. Root ends must extend beyond the wire baskets into the fill material. Place soil over the cuttings and compact it. Repeat the construction sequence until the structure reaches the required height.

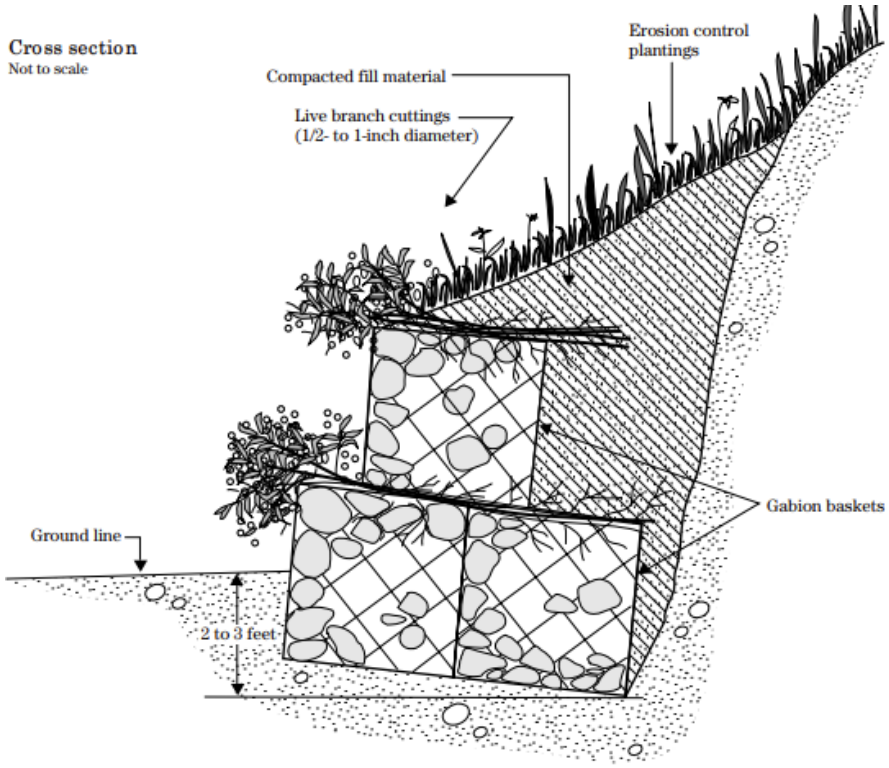


Figure 3.3.2.2-B: Vegetated gabion details. Source: NRCS (1996).

Joint plantings, sometimes referred to as vegetated riprap, are a bioengineered soil technique that involves tamping live cuttings into soil between the joints or open spaces in conventional riprap that is already set in place. Joint planting is ideal where riprap is required. This technique aids in soil moisture removal, preventing soil from washing out below the riprap, and in slope stability. Cut live branches one-half to one and one-half inches in diameter and long enough to extend into the soil below the riprap. Remove side branches from cuttings while leaving the bark intact. Tamp live branch cuttings into riprap void spaces during or after construction. Ensure the root ends extend into the soil beyond the riprap (Figure 3.3.2.2-C). Mechanical probes may be needed to create pilot holes for the live cuttings. Place cuttings orthogonal to the slope with growing tips protruding from the finished face of the riprap.



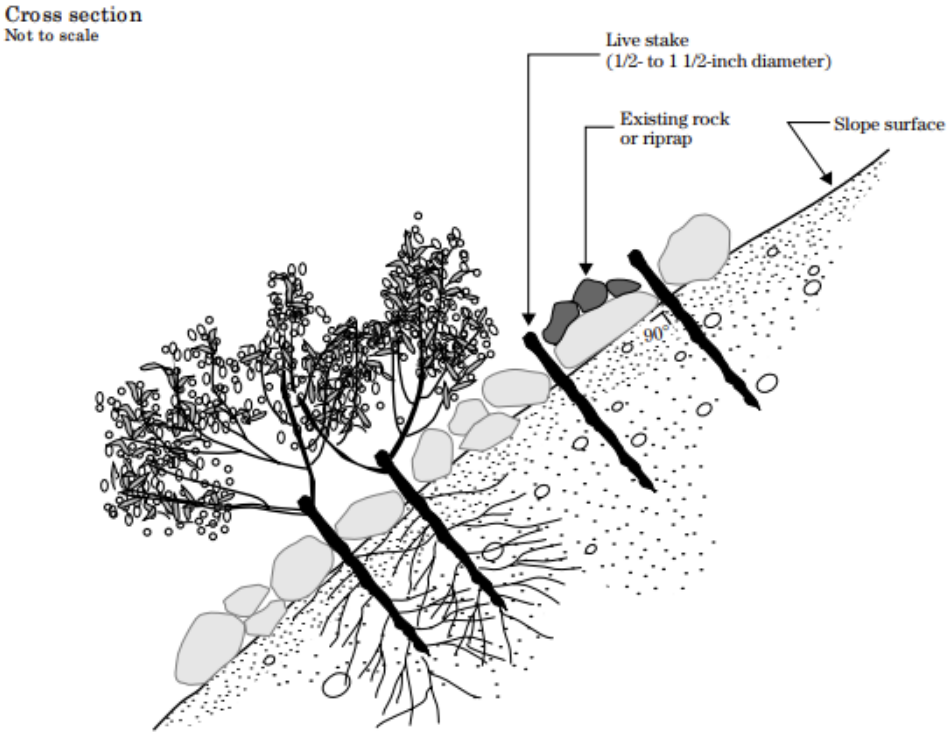


Figure 3.3.2.2-C: Joint planting details. Source: NRCS (1996).

Vegetated mechanically stabilized earth walls are soil retaining bioengineered structures designed to stabilize steep slopes while incorporating vegetation to improve environmental benefits. These walls use a porous fascia and tensile inclusions (reinforcements) integrated into the soil to distribute loads and resist lateral forces. Two common types are geocell systems and wrapped soil systems, each utilizing different materials for the fascia and reinforcements. Geocells contain soil and vegetation, with perforated cells allowing for root growth, while wrapped soil systems use geosynthetic fabric to hold compacted soil layers and support vegetation. The design of these vegetated mechanically stabilized earth walls requires careful consideration of factors such as slope, wall height, material selection, and geotechnical analysis. These walls can be constructed with various slopes, typically ranging from nearly vertical to 1H:1V, and may incorporate vegetation through seeding or live cuttings, depending on the project's needs. A key concern is ensuring the stability of the structure, particularly with respect to shear stresses and erosion protection. Walls are often reinforced with a stone toe to prevent undermining from scour. The structure must be engineered to handle external forces such as overturning, sliding, and soil pressure while ensuring proper drainage and vegetation growth. Design details are presented in Figure 3.3.2.2-D. Consult geotechnical experts when necessary, as factors such as shear stresses, erosion protection, and reinforcement placement are crucial for stability.


Source: TDOT

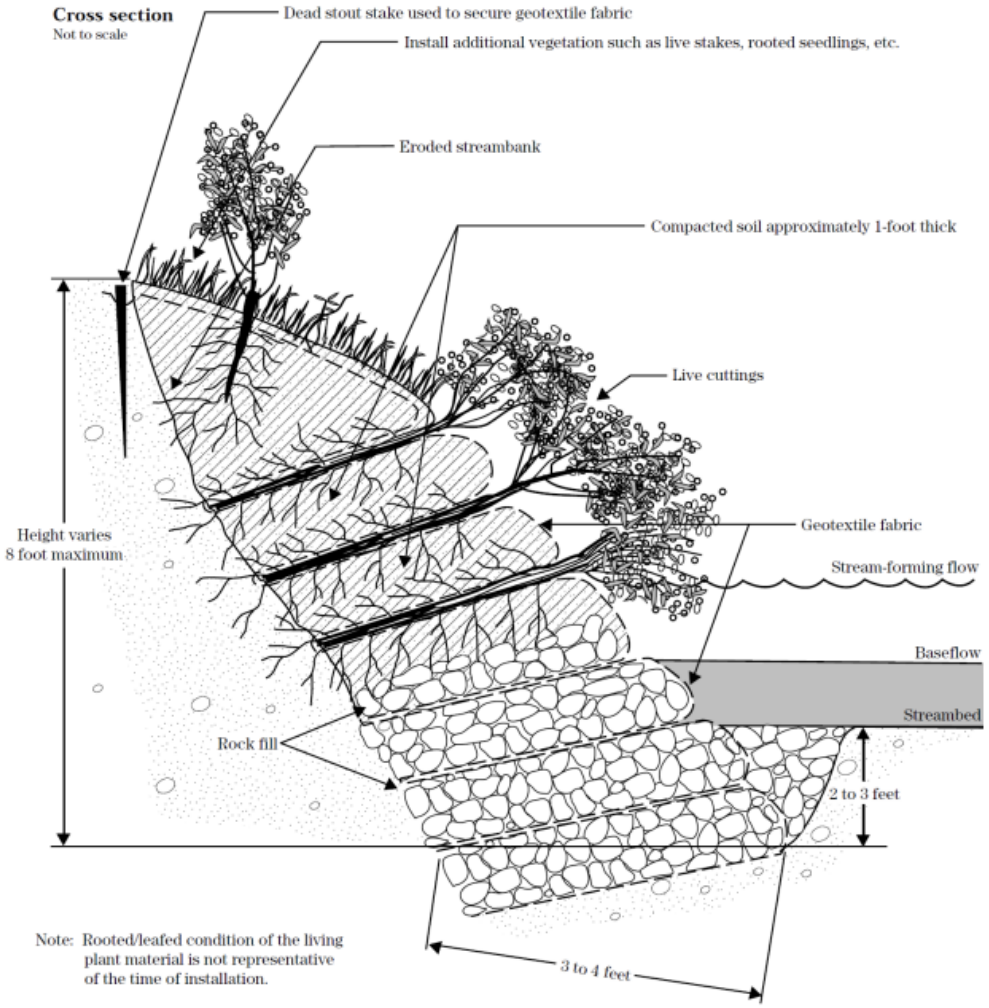


Figure 3.3.2.2-D: Design details of mechanically stabilized earth walls. Source: NRCS (1996).

Geotextile containments are bioengineered soil techniques that are relatively novel and commercially available products. The containment systems vary broadly in design within manufacturers and from manufacturer to manufacturer because of their broad applicability on bank slopes, lake perimeters, coastlines, dunes, etc. Typically, geotextile containments are to be staked below the finished grade at both the top and bottom of the bank. Containments can be filled with eroded sediment, soil fill, etc., and planted to ensure long-term stability. Refer to manufacturer specifications on stake material, stake spacing, alternative tie-down methods, tensile strengths, etc. All geotextile containments must in installed according to such specifications.

Soil bioengineering techniques use live, woody vegetative cuttings to increase slope stability. It can either be a woody vegetation system alone or a woody vegetation system combined with simple inert structures. Beyond structural benefits, these systems enhance riparian habitats, improve water quality, and offer aesthetic and ecological advantages (ALSWCC, 2018). Native plant species that root easily, such as willow, are ideal for soil bioengineering projects. Use plants that are well-suited to the conditions of each site and sourced locally, either from natural stands or reputable nurseries. However, careful consideration must be given to local wildlife interactions, as some species may pose challenges. For example, in west Tennessee, beavers are particularly drawn to willows, often undermining their effectiveness in stabilizing streambanks by causing additional erosion. When harvesting plant materials, cuttings can range from one-half to two inches in diameter and two to six feet in length, depending on project needs. To ensure regrowth, cut plants at a blunt angle, leaving an eight- to ten-inch trunk above the ground. Once harvested, cuttings are to be bundled, stripped of side branches, and kept moist to preserve viability. Careful handling during transport is essential to prevent damage and drying. It is ideal for cuttings to be delivered to the construction site within eight hours of harvest and installed immediately, particularly in temperatures above 50°F. If short-term storage is necessary, submerge cuttings in water or place them in moist, shaded soil to maintain their integrity for up to two days. For successful establishment, install cuttings during the dormant season, typically from late September to March. Soil conditions must support healthy plant growth, with compacted backfill ensuring proper contact between cuttings and the soil to eliminate air pockets. Thoughtful planning and careful handling of plant materials are essential for a resilient, long-lasting bioengineered streambank. Common slope stability techniques include live staking, live fascines, brushlayering, branchpacking, and brushmattresses.

Live staking is a slope stability technique that involves inserting and tamping live, rootable vegetative cuttings directly into the ground. As these stakes establish, they develop a dense root mat that binds soil particles, reinforces streambanks, and extracts excess moisture, making them particularly effective for stabilizing wet areas and repairing small earth slips. This method is cost-effective, quick to install, and can be used to secure surface erosion control materials or enhance vegetation colonization. Proper preparation includes removing side branches while keeping the bark intact, cutting the bottom end at an angle for easier insertion, and ensuring that the buds face upward. Stakes, typically one-half to one and one-half inches in diameter and two to three feet long, are spaced two to three feet apart in a triangular pattern, with four-fifths of each stake embedded in the soil (Figure 3.3.2.2-E). In firm ground, a pilot hole can be made with an iron bar, and stakes can be driven in using a dead blow hammer. To ensure effectiveness, compact the soil tightly around each stake, and replace any split or damaged stakes.

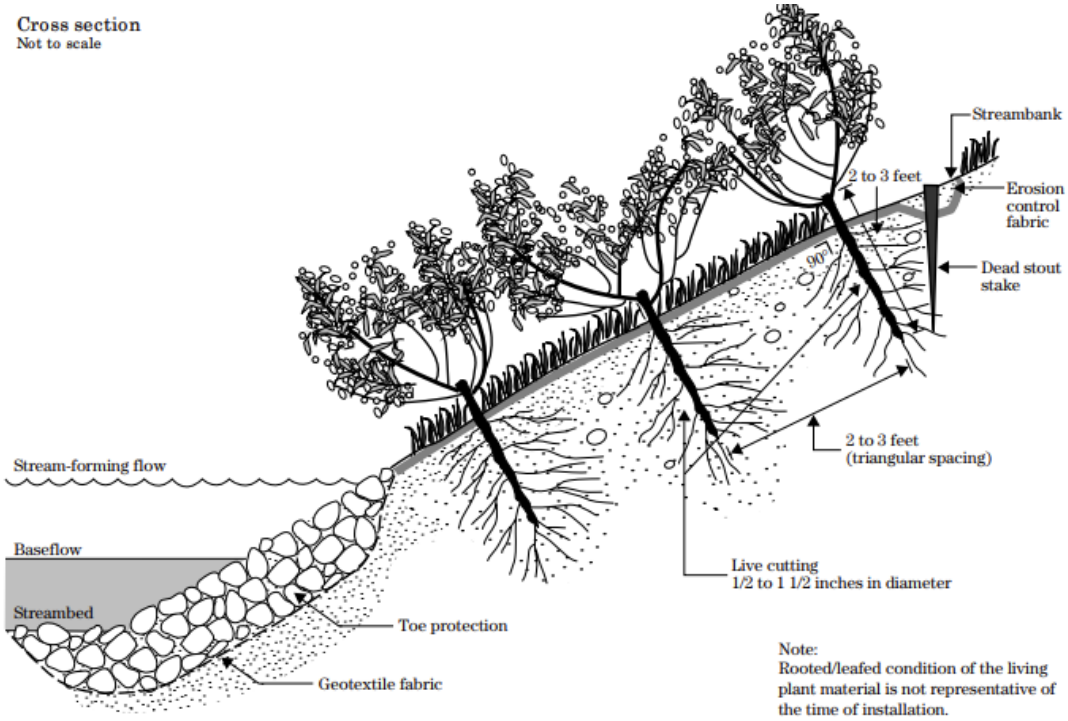


Figure 3.3.2.2-E: Live staking details. Source: NRCS (1996).

Live fascines are a slope stability technique involving cylindrical bundles of branch cuttings strategically placed in shallow trenches that can be ideal for steep or rocky slopes. These structures create a series of shorter slopes that minimize surface runoff and prevent shallow sliding. Typically composed of easily rootable species like young willows or shrub dogwoods, fascines also enhance drainage and encourage the natural colonization of vegetation. When properly installed, they provide immediate surface protection while gradually integrating into the landscape, reinforcing soil stability, and fostering plant growth. Construction involves tying cuttings into bundles, usually six to eight inches in diameter and five to 30 feet long, ensuring that growing tips align in one direction. The bundles are placed in trenches dug along the contour of the slope, secured with untreated twine and dead stout stakes driven at regular intervals. Live stakes are positioned downslope to promote additional root development (Figure 3.3.2.2-F). Proper soil contact is essential, with moist soil packed around the bundles, leaving them slightly exposed to facilitate rooting. Erosion control materials such as jute mesh or coconut netting may be added between rows for extra reinforcement. Over time, the fascines trap sediment, stabilize the soil, and create a microclimate conducive to long-term vegetation establishment.

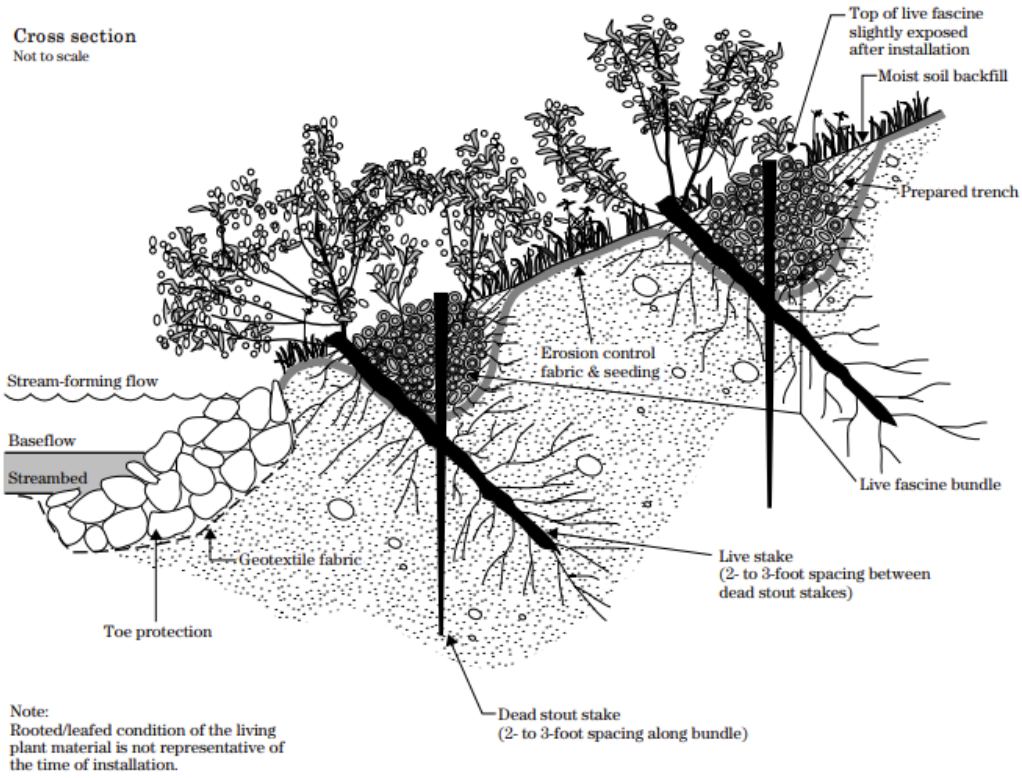


Figure 3.3.2.2-F: Live fascine details. Source: NRCS (1996, 2021).

Brushlayering is a slope stabilization technique that involves embedding live branch cuttings into excavated benches on a slope to provide stability and erosion control. Unlike live fascine systems, brush layering positions the cuttings perpendicular to the slope contour, allowing the protruding branch ends to disrupt surface runoff and reduce erosion. This method is particularly effective on slopes up to 2H:1V in steepness but no more than 15 feet in vertical height. The process begins at the toe of the slope, where horizontal benches, typically two to five feet wide, are excavated on contour or slightly angled downward to facilitate drainage. The benches are sloped inward to direct water away from the slope face. Branch cuttings, ideally from species that root easily, such as willows or shrub dogwoods, are arranged in overlapping or crisscrossed patterns with their basal ends embedded in the back of the bench. A layer of soil, sourced from the next upslope bench, is placed over the root ends, compacted to remove air pockets, and built up in six to eight-inch lifts until the desired thickness is achieved. Ensure the growing tips of the cuttings extend beyond the slope face


Source: NRCS (2021)

to help trap sediment and aid vegetation establishment. This technique provides immediate soil reinforcement through the unrooted stems while offering long-term stabilization as the cuttings take root and develop into mature vegetation. The live stems act similarly to geotextiles, increasing frictional resistance and preventing shallow slides. Brushlayering also serves multiple ecological functions, including breaking up long slopes into smaller segments to slow surface runoff, trapping debris, adjusting microclimates for seed germination, and redirecting slope seepage. On slopes with a gradient of 3H:1V, long straw or similar mulch can be applied between brush layers, while steeper slopes require additional reinforcement with jute mesh or other erosion control materials. Over time, the rooted branches integrate into the slope, enhancing stability, improving infiltration, and fostering the natural regeneration of vegetation.

Branchpacking is a slope stability technique that involves layering live branches with compacted soil to repair small slumps or holes in slopes. This method is particularly effective for reinforcing earth in small fill sites where erosion control and mass stability are needed. Typically used on areas no deeper than four feet and no wider than five feet, branchpacking relies on the mechanical reinforcement provided by both the branches and wooden stakes. The process begins by driving wooden stakes, ranging from five to eight feet in length, vertically into the ground at one to one and a half foot intervals. The lowest point of the slump is treated first. A layer of live branches, about three to six inches thick, is placed between the stakes, with their growing tips extending slightly beyond the face of the slope, while the root ends touch the undisturbed soil at the back of the hole. Each layer of branches is followed by a layer of compacted soil to ensure close contact between the cuttings and the backfill. This alternating process continues until the hole is completely filled, and the final installation blends seamlessly with the natural slope (Figure 3.3.2.2-G). Branchpacking provides immediate reinforcement to the soil while promoting long-term stability as the live cuttings take root and establish vegetation. The protruding branches serve to slow surface runoff, trap debris, and reduce the risk of shallow slides by increasing friction within the slope. Over time, as the branches develop into rooted plants, they enhance the slope's resistance to shear displacement and further strengthen the embankment. This technique is not suitable for repairing larger slumps but is highly effective for stabilizing minor earth movements and maintaining slope integrity. To maximize effectiveness, keep the soil moist throughout the installation to prevent drying of the live cuttings and properly trim the terminal buds to encourage lateral sprouting.

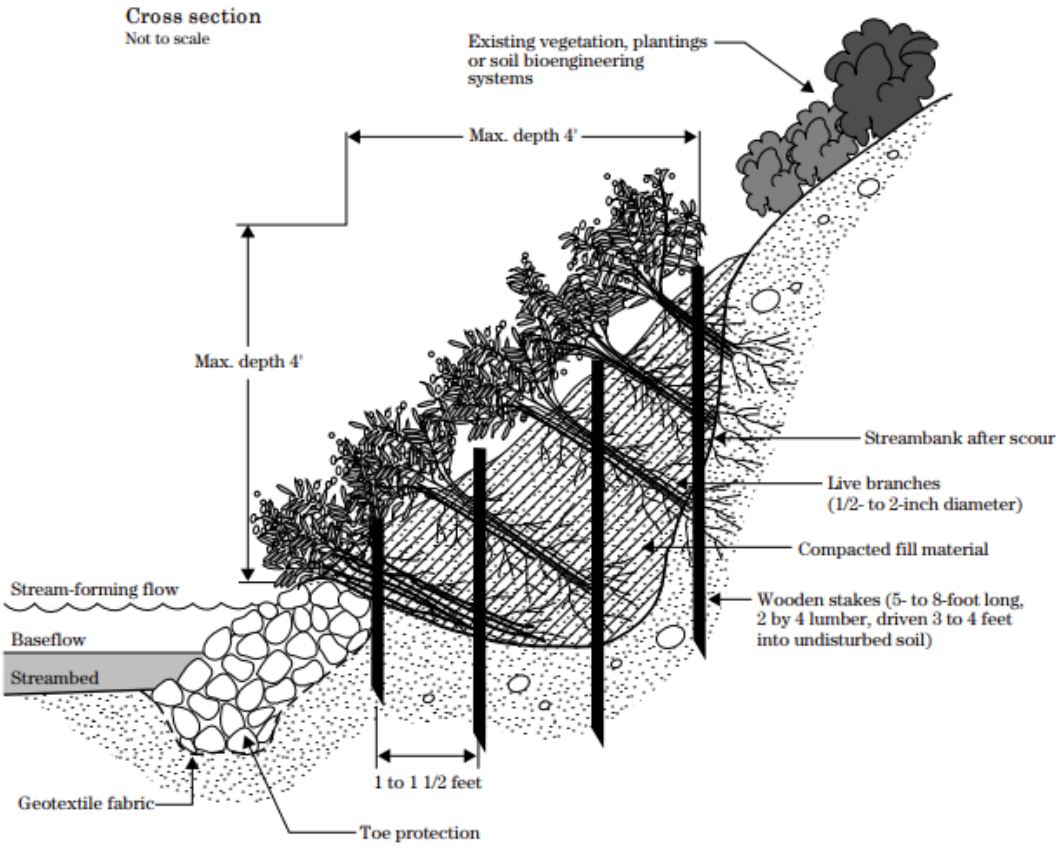


Figure 3.3.2.2-G: Branchpacking details. Source: NRCS (1996, 2021).

A brushmattress is a slope stability technique that integrates live stakes, live fascines, and branch cuttings to stabilize streambanks. Installed from the base of the slope upward, it provides immediate erosion control, making it particularly useful for steep, fast-flowing streams. Beyond structural reinforcement, it captures sediment during floods, rapidly restores riparian vegetation, and fosters conditions for native plant colonization, enhancing streamside habitat. The construction of a brushmattress requires flexible branches, typically six to nine feet long and about one inch in diameter, secured with untreated twine and wire. Installation begins by grading the unstable bank to a gradual slope, followed by excavating a trench at the base to anchor the vegetation. Live and dead stout stakes are driven into the slope in a uniform grid to secure the brush layer, with live fascines placed over the basal ends of the branches (Figure 3.3.2.2-H). Thin layers of soil are added between the branches to encourage rooting while leaving the surface slightly exposed. This method not only reinforces the streambank but also accelerates ecological recovery, blending natural function with long-term stability.

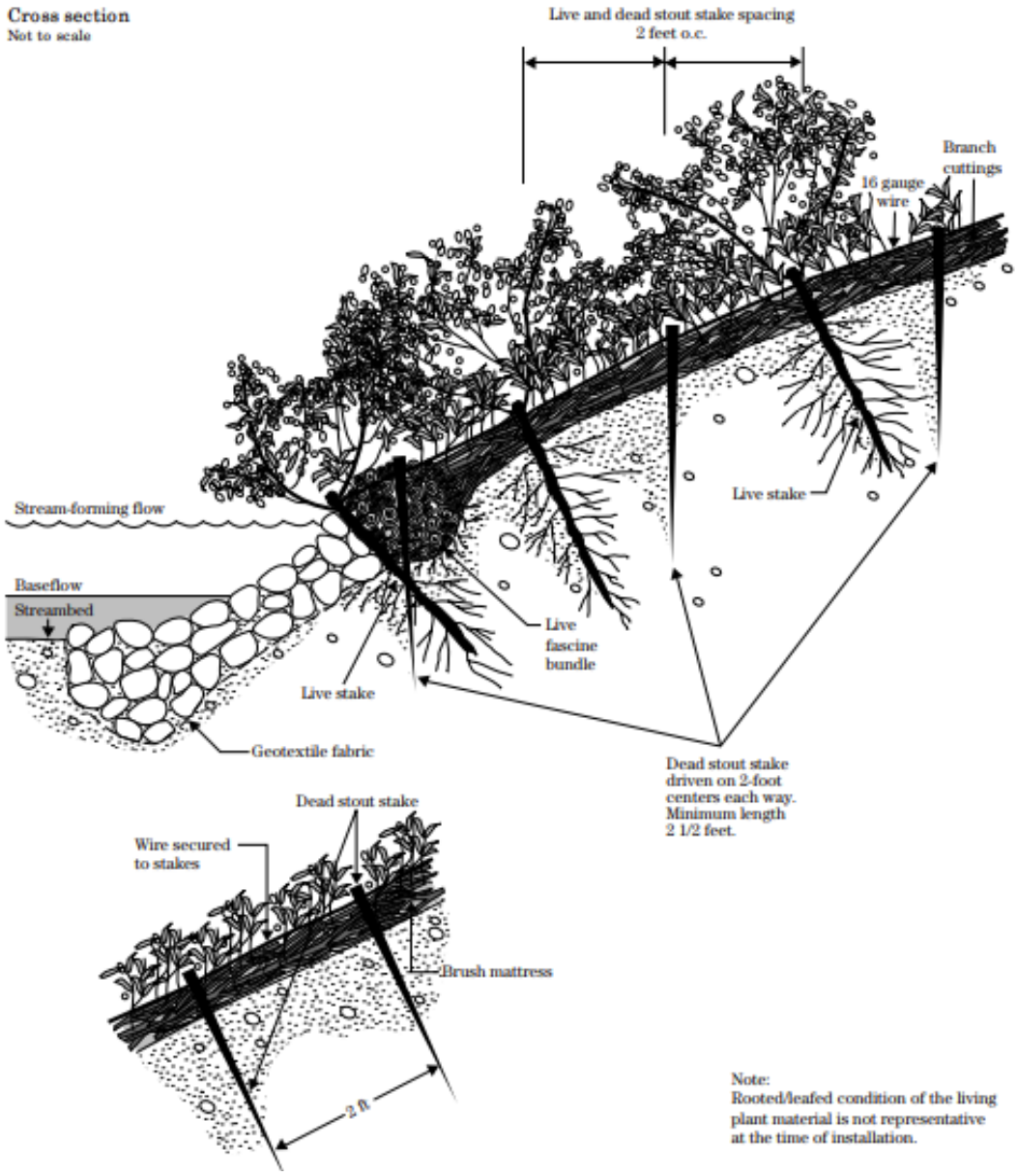


Figure 3.3.2.2-H: Brushmattress details. Source: NRCS (1996).

Designers are referred to NRCS (1996, 2021) for additional bioengineered soil bank stability and slope stability techniques if none of the presented material herein are suitable or desirable for specific project constraints. Though less commonly implemented, other techniques presented in the referenced material are allowable for use in construction sites with coverage under the Tennessee CGP.

Example Application

-Example courtesy of TDOT-

Given:

A channel bank approximately 6 feet high and 50 feet long is experiencing bank stability problems. The stream has a bedrock channel bed, and hydraulic analysis indicated that the channel velocity in the 50-year flood event is 7.23 ft/s.

Determine:

The quantity of vegetated gabions for this project.

Solution:

Utilizing the Geotechnical Engineering Section in TDOT, it has been determined that sufficient structural integrity can be achieved by stacking two rows of gabions to form a near-vertical wall. To meet the 6-foot height of the channel bank, it is determined that the wall should be constructed with 3 × 3 × 6 ft gabion baskets. Further, lateral stability will be ensured by cutting a shallow groove into the exposed basket to serve as a foundation for the gabion wall.

Based on the hydraulic analysis for the 50-year event, it is determined that machined riprap Class B would be required (Tables 3.3.2.1-A and 3.32.1-C). Although the velocity appears somewhat high for the structure of gabions, it is anticipated that the presence of vegetation along the structure will reduce flow velocities and thus prevent abrasions on the wire coatings.

Each layer of the wall will be constructed by placing 3 x 3 x 6-foot gabions end to end. Thus, 9 gabions will provide a length of 54 ft, and an additional gabion will be placed to provide a sufficient length of erosion protection past the end of the curved section of the channel alignment (60ft). Thus, the wall will contain 20 gabions, each with a volume of 54 ft^3 for a total volume of 1080 ft^3 or 40 cubic yards. The geotextile will be placed across the top and down the inside face of the wall. In order to provide additional protection against the piping of fill through the wall, an additional length of 3 ft will be placed on the bedrock at the toe of the wall to form an apron. Thus, the total surface area of the geotextile may be computed as:

$$A = W \times L = (3 + 6 + 3) \times 60 = 720 \text{ ft}^2 = 80.0 \text{ square yards.}$$

Erosion Prevention and Sediment Control Handbook

-Example courtesy of TDOT-

Given:

Vegetated riprap is proposed on a 200-foot reach of a small stream. The stream has a bottom width of 5 feet and 1.5H:1V side slopes. Hydraulic analysis of the stream for the channel-forming discharge yields a flow depth of 2.5 feet and a velocity of 3.7 ft/s. The hydraulic analysis for the 50-year event yields a flow velocity of 5.66 ft/s.

Determine:

The quantity of vegetated riprap for this project.

Solution:

Based on the hydraulic analysis for the 50-year event, it is determined that machined riprap Class B would be required (Tables 3.3.2.1-A and 3.3.2.1-C). Based on TDOT standard specifications, the minimum depth of the riprap layer should be 2.5 feet.

The height along the slope over which riprap is to be installed will be equal to 1.5 times 2.5 feet, which is 3.75 feet. The quantity of vegetated riprap for each side of the channel can be determined by computing the required volume of riprap as follows:

$$V = H \times D \times L = 3.75 \times 2.5 \times 200 = 1875 \text{ ft}^3$$

Thus, the total volume for both sides of the channel will be 3750 $\text{ft}^{3,}$ which can be divided by 27 to determine the final quantity of 139 cubic yards.

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3.3.2.3 Temporary Instream Diversion



Source: TDOT

Definition and Purpose

Instream diversions use temporary dikes to redirect stream baseflow, allowing for construction activities such as multi-barrel box culverts, bridges, bank stabilization, structural repairs, and intake or outfall installations to occur without disrupting flow. This method typically channels water into an existing or newly built barrel, keeping other construction areas dry. By isolating the work zone from the active flow of water, instream diversions help minimize sediment transport, protect water quality, and provide aquatic organism habitat throughout construction.

Appropriate Applications

Instream diversions are practical solutions for projects requiring instream work, particularly when constructing multiple barrel structures. When properly phased, they can also accommodate projects where roadway traffic must be maintained during construction (TDOT). Where applicable, an instream diversion may be more cost-effective than a temporary diversion channel or a temporary diversion culvert.

Limitations and Maintenance

Instream diversions are designed to convey low flow conditions as opposed to a design event like temporary channels or culverts. As a result, they are more susceptible to overtopping during rainfall. When selecting a diversion method, evaluate both the duration of

construction and the potential impacts of flooding within the work area. Dewatering practices used for the diversion will need to be maintained to function as intended (Section 4.4.12.2).

Installing instream diversions in jurisdictional waters requires additional permits, such as an ARAP, and therefore, both the conditions of the CGP and ARAP must be followed. A Section 404 permit from USACE may also be required, and if the proposed instream diversion is to be completed in a TVA reservoir, a TVA 26a permit may also be necessary. Local permitting may be required in addition to state and federal permitting requirements. Consider the criteria and conditions of the necessary permits during the planning stages of the project and EPSC plans.

Planning and Design Considerations

Effective instream diversions require careful sequencing to ensure both regulatory compliance and functional efficiency. The symbol and detail can be shown in the plan set with generalized sequencing notes. These notes not only assist TDEC reviewers in verifying compliance with the CGP but also establish a foundation for more detailed sequencing notes once a contractor is selected. After a contractor is hired, refine and update these notes as necessary on the plan sheets to reflect the specific construction approach. Ultimately, the permittee bears responsibility for ensuring that all permit conditions are met before approving the work.

To ensure effective flow management, set the diversion elevation at least one foot above the baseflow elevation within the reduced channel width of the stream (TDOT). Though not required in the state of Tennessee, some states implement side slope limitations of the diversion to 2H:1V (ALSWCC, 2018; GSWCC, 2016). For projects requiring uninterrupted roadway access during construction, the engineer may need to extend the overall length of the box structure to accommodate traffic considerations.

To assess whether the instream diversion can adequately accommodate baseflow under various channel configurations, the following methodology is recommended (TDOT):

1. *Estimate flow characteristics*: Determine the typical width and depth of flow within a representative section of the stream under normal conditions. This can be achieved using field measurements, aerial imagery, or site topographical data.
2. *Calculate cross-sectional flow area*: If the channel is rectangular, multiply the estimated flow depth by the stream width to compute the total cross-sectional flow area. For other channel configurations, refer to Table 2-A.
3. *Determine diverted flow depth*: Identify the width of flow through the diversion channel and divide the original cross-sectional area by this new width to estimate the depth

of flow in the diverted section, if rectangular. For other geometries, refer to Table 2-A.

4. *Verify wetted perimeter*: Compute and compare the wetted perimeter of both the natural stream cross-section and the diverted section (Table 2-A). If the diverted section exhibits a greater wetted perimeter, it indicates excessive channel narrowing. In such cases, adjust the design to provide a wider flow width to prevent excessive flow constriction.
5. *Establish final diversion height*: Set the final elevation of the diversion at least one foot above the newly determined diverted flow depth to ensure adequate containment.

Hydraulic computations for each channel shape are governed by channel geometries (Table 2-A), the Continuity Equation (Eqn 7), and Manning's Equation (Eqns 6 and 8). However, tools can be used to reduce computation time, ensure accurate computations, and/or for less typical cross sectional shapes (Fang, 2007; Mejia & Ponce; Perez et al. 2015) as specified in Section 2.1.2.

Where possible, design the sinuosity of the diverted section to mimic the natural flow path of the stream to minimize hydrologic disruption, such that the diversion length is greater than or equal to the length of the diverted stream portion (ALSWCC, 2018). If excavation is required to achieve the necessary flow dimensions, consider alternative diversion methods rather than expanding the existing stream channel, as widening may introduce unnecessary environmental impacts.

A variety of materials can be employed in constructing instream diversions, with polyethylene sheeting (minimum six mil thickness) commonly incorporated to enhance effectiveness. For instance, integrating polyethylene sheeting within a sandbag berm or draping it over jersey barriers can help maintain flow separation and prevent seepage into the work area.

Conduct dewatering using a pump system once the instream diversion is in place and has successfully isolated the work area from the active flow. Effluent from this operation must be properly managed by directing it through a sediment filter bag (refer to Section 4.4.12.6) or a designated dewatering structure (refer to Section 4.4.12.2) to prevent sediment discharge and maintain water quality standards.

Example Application

-Example courtesy of TDOT-

Given:

An existing slab bridge on vertical abutments with a span of 20 feet is to be replaced by a box bridge with three 12-foot spans. A temporary run-around will be used to

maintain traffic so that the structure can be replaced without traffic. The designer has determined that an instream diversion may be used at this site.

Determine:

The height for the instream diversion.

Solution:

Step 1- Assume a phasing plan: Although it is the responsibility of the contractor to devise a phasing plan, the designer will need to make an educated guess of the plan in order to estimate lengths for each section of the instream diversions.

Based on the bridge layout plan, it appears that the channel will be shifted somewhat to the west in order to accommodate the increased width of the proposed structure. Thus, it is assumed that the phase 1 instream diversion will extend from the west bank of the channel to the west side of the existing structure. This will maintain flows through the existing structure while allowing the western most barrel of the proposed structure to be built. In phase 2, flows would be diverted into the new barrel of the proposed structure by extending instream diversions from the east bank of the channel to the east side of the new barrel. This will allow the demolition of the existing structure and the construction of the two remaining barrels.

Step 2- Determine heights of the instream diversions: Aerial photography of the site is examined in order to estimate the average water surface width of the stream in the project area. This is found to be 25 feet. In this same area, the project survey data indicated that the flow line of the stream is about 1.2 feet below the elevation at the edge of the water. Thus, the cross-sectional flow area (A) and wetted perimeter (P) of the normal flow are computed following Table 2-A:

$$A = 25 \times 1.2 = 30 \text{ ft}^2$$

$$P = 25 + 2 \times 1.2 = 27.4 \text{ ft}$$

In phase 1, the width of flow in the diverted section will be 20 feet. Since the cross-sectional area of the flow in the diverted section is assumed to be equal to the natural cross-sectional flow area, the depth in the diverted section (d_d) is estimated as:

$$d_d = 30 / 20 = 1.5 \text{ ft}$$

Based on this depth, the wetted perimeter of the flow in the diverted section is computed following Table 2-A:

$$P = 20 + 2 \times 1.5 = 23 \text{ ft}$$

Since the wetted perimeter in the diverted section is less than in the natural section, the flow width will be adequate. Thus, the required minimum height of the instream diversion will be 2.5 feet.

In phase 2, the width of the diverted section will be reduced to 12 feet. Using the same process as described above, the depth in the diverted section is found to be 2.5 feet, and the wetted perimeter is found to be 17 feet. Again, the wetter perimeter in the diverted section is found to be less than that in the natural section, and the height of the instream diversion will be 3.5 feet.

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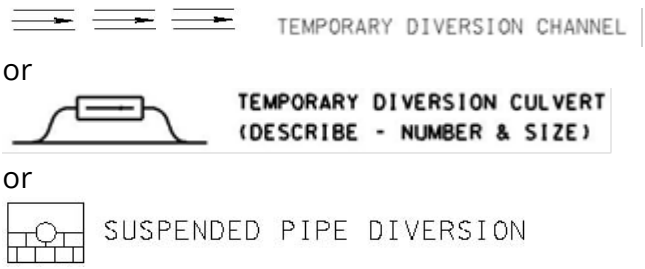
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TDOT. *Drainage Manual Ch10*.

3.3.2.4 Temporary Stream Diversion





Source: TDEC

Definition and Purpose

Temporary stream diversions are implemented to redirect baseflow from active construction areas, such as bridges and culverts, allowing work to proceed in dry conditions. By isolating construction from the active flow of water, these diversions effectively minimize sedimentation and protect water quality, reducing potential environmental impacts associated with in-stream disturbances.

Appropriate Applications

Stream diversions are used where work in the stream is unavoidable and where the diversion will not cross an existing roadway where traffic is maintained (TDOT). This measure is best suited in low flow conditions. Often, a temporary stream diversion is used for smaller channels where baseflow can be entirely contained in a manmade channel or conveyed through a pipe temporarily.

Limitations and Maintenance

ALSWCC (2018) and TDOT suggest temporary diversions be constructed in drainage basins smaller than one and two square miles, respectively. Instream diversions may be better suited for larger streams (e.g., greater than one or two square miles) where conveying baseflow is more challenging.

Installing instream diversions in jurisdictional waters requires additional permits, such as an ARAP, and therefore, both the conditions of the CGP and ARAP must be followed. A Section 404 permit from USACE may also be required. If the proposed diversion is to be completed

in a TVA reservoir, a TVA 26a permit may also be necessary. Local permitting may be required in addition to state and federal permitting requirements. Consider the criteria and conditions of the necessary permits during the planning stages of the project and EPSC plans.

Inspect stream diversion channels at the end of each day to ensure flow control measures and construction material are positioned and adequately secured, thereby eliminating material waste downstream. All repairs should be made immediately. Dewatering practices used for the diversion will need to be maintained in order to function as intended (Section 4.4.12.2). Finally, stream diversion channels should be constructed such that they do not impede upstream or downstream movement of aquatic organisms whenever possible.

The maintenance for the different temporary stream diversions is type-specific:

Bypass pumps require routine maintenance to remain effective as a stream diversion. Conduct daily inspections to ensure proper pump operation, check temporary piping for leaks, and repair any damage to the impervious dike. The discharge point must be monitored for potential erosion, and flow should be confirmed as adequately diverted through the pipe. Bypass pumping may not be suitable when discharge locations cannot be stabilized, ponding to submerge the suction line is impractical, or the pump cannot handle normal flow within the stream.


Source: NCDOT (2003)

Suspended pipe diversions require regular inspections of the inlet and impervious dike to confirm no damage or leakage and to confirm proper flow diversion. Regularly clear sediment and debris from behind the dike and inlet to prevent blockages. The outlet must also be checked for potential erosion and to ensure the system is functioning effectively. Additionally, the inlet should be securely anchored and sealed to maintain stability.

Piped diversions require frequent inspections for any signs of damage or malfunction. Accumulated sediment and debris must be removed from the berm and inlet to maintain unobstructed flow. Monitor the outlet for erosion potential, and any diverted flow bypassing the temporary pipe must be addressed immediately to prevent surface erosion.

Fabric-lined diversion channels require regular inspection for stability under normal flow conditions and following all rainfall events. Signs of displacement or failure must be addressed promptly. It is essential to ensure that earthen material does not come into direct contact with the water body to prevent contamination and maintain water quality.


Source: TDOT

Planning and Design Considerations

All diversions must not impede aquatic organism passage. Disturbance within the confines of stream banks is required to be conducted during dry weather or separate from flowing water. The operation of excavation equipment is not to take place in flowing waters. Minimize the duration of the instream work as well as the clearance of the stream-bed and banks. During construction, all excavated material must be stabilized and stored outside the 100-year floodplain unless otherwise authorized. Any water pumped from the worksite must be discharged through an approved sediment filtration device, such as a sediment basin (Section 4.4.7) or filter bag (Section 4.4.12.6), to prevent contamination of downstream waters. Energy dissipation measures (Section 4.3.4) must be installed at the pipe outlet to prevent scour and erosion.

Common temporary stream diversions include bypass pumping, suspended pipe diversions, piped diversions, and fabric-lined diversion channels. Diversions that use impervious dikes or berms require methods to eliminate or reduce sediment from potential stormwater entering the construction area, such as pumping stormwater from the work area to a sediment filter bag (Section 4.4.12.6). Consider placing the EPSC measure outside of the designated buffer zone or at least the minimum buffer distance required in the CGP. Regardless of the type of stream diversion chosen, ensure the dimensions of the diversion can adequately convey the bankfull capacity of the stream (ALSWCC, 2018) or the required design flow, which is specified by either the CGP or local standards.

Use bypass pumping during low-flow conditions to minimize land disturbance. This method involves using a bypass pump along with an impervious dike or berm to redirect water from an upstream section to a downstream section through controlled pumping. Since this process transfers water directly from one point to another, it is crucial to discharge at a low flow rate to prevent erosion at the outlet. Bypass pumping is most effective for short-duration construction activities that do not require prolonged water diversion. Construction specifications include:

- Set up bypass pump and hose. Place the hose outlet in such a way as to minimize erosion at the discharge site or provide temporary energy dissipation measures. Consider anchoring the pump and hose;
- Construct outlet protection if needed for the diverted water so as not to create erosion;
- Construct an impervious dike or berm upstream of the work area to impound water for the bypass pump intake. Consider using a floating intake for pumps;
- Construct an impervious dike or berm downstream to isolate the work area;
- Check the operation of the pump system. Have an operation plan in place for when the pump will be run and who will manage it if it is not automatic; and
- Upon completion of construction, stabilize the work area in the stream and then remove the impervious dikes or berms, bypass the pumping system.

Temporary pipes are used to divert the small flows around the work area without the use of pumping operations. While the cost is higher for this operation than for an open plastic-lined channel, the probability of sediment loss is much lower than with an open diversion channel. Use this practice where adequate slope and space exist between the upstream and downstream ends of the diversion. The diversion pipe material should be high-density polyethylene (HDPE) or equivalent and extend at least one foot beyond the upstream and downstream toes of the diversion to provide adequate durability. Pipes must be properly secured using sandbag plugs at both the upstream and downstream ends, ensuring that the top of the plugs extends at least as high as the natural channel banks. If elbows are used to connect the pipe to the natural stream, they must be anchored using sandbags and sealed with polyethylene sheeting to create a watertight connection. A minimum cover of 24 inches must be maintained over the pipe to protect against structural damage and ensure stability. For circular pipes, the diameter of the pipe can be calculated from a reformulation of Manning's Equation (Eqn 15):

$$D = 16 \times \left(\frac{q_p \times n}{\sqrt{S}}\right)^{3/8} \quad \text{(Eqn 15)}$$

where D is the pipe diameter (inches), q_p is the peak flow rate (cubic feet per second), n is Manning's roughness coefficient (unitless), and S is the pipe slope (feet per foot). Round the computed pipe diameter up to the next common size. Rounding up the pipe diameter changes the peak flow capacity, hydraulic radius, wetted perimeter, etc., and therefore, also the peak velocity (V_P) in the pipe. Construction specifications include:

- Install a temporary pipe adjacent to the work area. Excavation may be required to provide a positive drainage slope from the upstream to the downstream side;
- Connect the downstream temporary pipe to the existing downstream channel. Place the outlet of the pipe to minimize erosion at the discharge site or provide temporary energy dissipation measures;
- Connect the upstream temporary pipe to the upstream existing channel;

- Construct an impervious dike or berm at the upstream side of the existing channel to divert the existing channel into the temporary pipe;
- Construct an impervious dike or berm at the downstream side of the bypass pipe to isolate the work area; and
- Upon completion of construction, stabilize the disturbed area, then remove the impervious dikes or berms and temporary pipe**.**

Use the suspended bypass pipe where an existing pipe or culvert is extended and is large enough to accommodate the bypass pipe. This bypass pipe is constructed inside the existing pipe or culvert to divert the watercourse through the construction area, thereby keeping the work area dry. Construction specifications include:

- Install a temporary pipe through the existing pipe or culvert to be extended. Place the outlet of the temporary pipe to minimize erosion at the discharge site or provide temporary energy dissipation measures;
- Construct an impervious dike or berm upstream of the work area to divert flow through the temporary pipe. Anchor and seal the temporary pipe securely at the inlet;
- Construct an impervious dike or berm at the downstream side of the bypass pipe to isolate the work area; and
- Upon completion of the culvert or pipe extension and once the work area is stabilized, remove the impervious dikes or berms and the temporary pipe.

A fabric-lined temporary diversion channel is used to divert normal stream flow and small storm events around the work area without the use of pumping operations. The temporary diversion channel is typically constructed adjacent to the work area and is stabilized by lining with a geotextile fabric (Type III) to minimize the potential for erosion within the temporary diversion channel (TDOT). Ensure the selected lining can withstand design velocities per Tables 3.3.2.1-A and 3.3.2.1-B, as well as per manufacturer specifications (ALSWCC, 2018). Geotextile linings alone may not be sufficient to prevent erosion. In cases where design velocities exceed two and a half feet per second, consider the use of conventional riprap or other structural linings in addition to geotextiles (TDOT). Use this practice where adequate space and slopes exist adjacent to the work area. Avoid creating steeper slopes than preexisting natural conditions. Consider alternative methods in summer months, as plastic linings can thermally enrich stream water. Though this is not a CGP requirement, many aquatic species in Tennessee are sensitive to thermal fluctuations. Construction specifications include:

- Excavate the diversion channel without disturbing the existing channel. Ensure the dimensions of the channel can convey the design flow. Refer to Section 3.3.2.3 to adequately size a channel;

- Place poly-fabric liner in the diversion channel with a minimum of four feet of material overlapping the channel banks. Secure the overlapped material using at least one foot of fill material. Ensure the liner and fill material can withstand the velocity of water during the design flow. Refer to Section 4.2.6.1 to adequately line a channel;
- Connect the downstream diversion channel into the downstream existing channel and secure the poly-fabric liner at the connection;
- Connect the upstream diversion channel into the upstream existing channel and secure the poly-fabric liner at the connection;
- Construct an impervious dike or berm in the existing channel at the upstream side to divert the flow into the diversion channel;
- Construct an impervious dike or berm in the existing channel at the downstream side to isolate the work area and capture any construction stormwater;
- Dewater the work area of any construction stormwater through a sediment filter bag or similar, as needed;
- Upon completion of construction and stabilization of the work area, remove the impervious dikes or berms and divert the channel back into the newly constructed reach or culvert;
- Remove the poly-fabric liner and fill in the diversion channel to the previous grade unless new elevations are shown on the plans; and
- Establish vegetation on all disturbed areas.

Upstream and downstream transition lengths should be one and one and a half times the channel width, respectively, when the ratio of the radius of curvature to channel width is greater than or equal to three. When the radius of curvature to channel width ratio is less than three, utilize Figure 4.2.3-A to determine transition lengths. When using Figure 4.2.3-A, the hydraulic radius, r, can be calculated utilizing Fang (2007), as specified in Section 2.1.2.

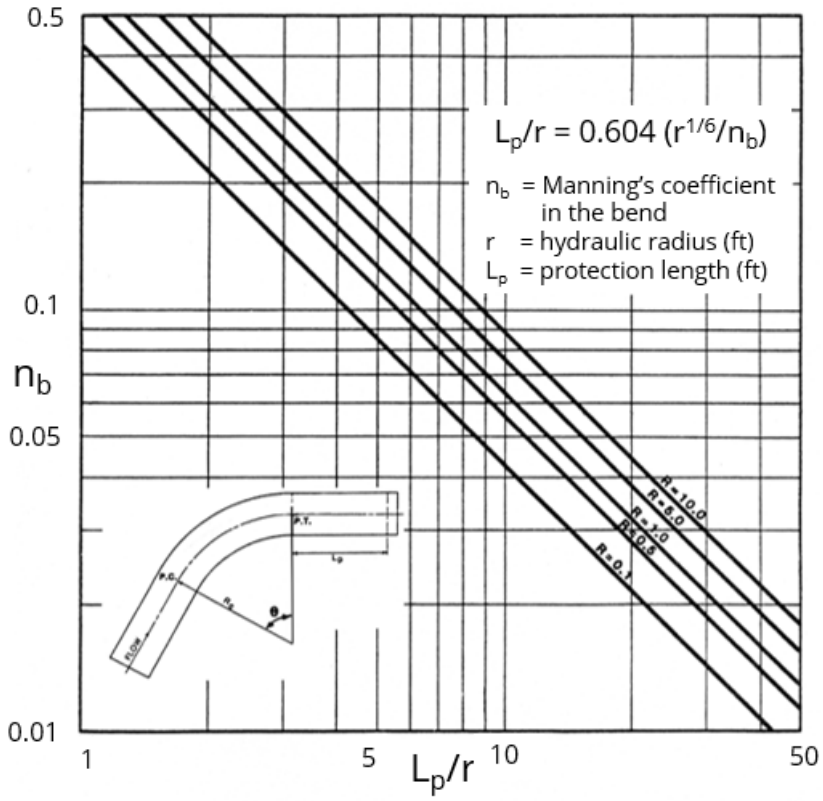


Figure 4.2.3-A: Protection Length (L_P) downstream of a channel bend, in this case, a channel transition for a diversion. Amended from: TDOT.

Example Application

For designing a fabric-lined diversion channel, refer to Section 3.3.2.3 for an example regarding the sizing of the channel and Section 4.2.6.1 for lining the channel.

-Example courtesy of TDOT-

Given:

A firm is designing a culvert replacement on an existing roadway that crosses a small stream in rural southwest Tennessee, Fayette County (CN=75). The roadway consists of two 12-foot lanes, with six-foot shoulders. The replacement structure will consist of a 12-foot × 4-foot box culvert, 100 feet long. In order to allow this project to be constructed in the dry, a temporary diversion culvert will be used. The drainage area at the site is 120 acres, with a time of concentration of 0.70 hours, and the stream slope is 0.5%. The area is Hydrologic Area 4.

Determine:

The required design and quantities for a temporary diversion culvert.

Solution:

Step 1- Determine the design flow rate: Though temporary diversions are typically designed for short-term use and not intended to convey stormflow, for the purposes of this example, it will convey stormflow. The design flow rate should be based on the 2-Year, 24-hour storm. Since the drainage area at this site is greater than 100 acres, the TR-55 Graphical Peak Discharge Method will be used (Eqn 10, Section 2.1.3). The design flow rate is computed as follows:

- 2-year, 24-hour rainfall: P = 3.98 in (NOAA)
- Initial abstraction: I_a = 0.667 in (Eqns 2 and 3, Section 2.1.1)
- Compute I_a / P = 0.1679
- Unit peak discharge coefficient, q_u = 440 csm/in (NRCS, 1986)
- Runoff depth, Q_{CN} = 1.653 inch (Eqn 2, Section 2.1.1)
- Assuming a ponding factor, F_p =1, the peak discharge, q_p, is calculated to be 136.4 cfs (Eqn 10).

Step 2- Select the culvert size and diversion type: The smallest single (circular) pipe capable of conveying the 2-year flow rate can be calculated with Eqn 15.

$$D = 16 \times \left(\frac{q_p \times n}{\sqrt{S}}\right)^{3/8}$$

$$D = 16 \times \left(\frac{136.4 \times 0.024}{\sqrt{0.005}}\right)^{3/8} = 67.4 \text{ inches}$$

$$D = 72 \text{ inches} = 6 \text{ feet}$$

Since the site offers eight feet of clearance between the existing channel bottom and the profile grade of the roadway, there will be sufficient cover, and this size will be acceptable. Due to the diameter of the selected pipe, it is judged that the connection between the diversion culvert and the natural channel should be provided by means of excavated channel transitions, rather than by providing pipe elbows.

Step 3- Design the upstream channel transition: Based on examination of the plans, it is determined that the existing channel bottom is approximately six feet wide. Thus, the excavated channel transitions will also have a bottom width of six feet. It is also determined that the average depth of the channel transition will be four feet, with 2H:1V side slopes.

Based on inspection of the site layout, it is determined that this transition should be 30 feet long and that it should have a radius of curvature of 25 feet. Riprap erosion protection should extend past the end of the curved channel transition into the natural channel. Since the radius of curvature for this transition is more than three times the bottom width, the upstream riprap should extend 1.0 times the channel bottom width, or six feet from the end of the diversion channel. Thus, the total length of this transition will be 36 feet (refer to TDOT standards presented in Section 5.05.7

for these details). The procedure provided in Sections 3.3.2.1 – 3.3.2.3 of this manual is used to determine normal depth for the design flow rate in the natural channel. Based on a trapezoidal cross-section with a six-foot bottom width, 2H:1V side slopes, and stream slope of 0.005 ft/ft, it is found that the normal depth would be 2.86 feet, at a velocity of 4.07 feet per second. Since the velocity at the design flow rate is less than 5 fps, machined riprap Class A-1 will be suitable for this transition (Table 3.3.2.1-A).

Step 4- Design the downstream channel transition: The main portion of the proposed diversion channel will be 100 feet long in order to match the length of the proposed permanent structure. However, since the diversion channel is located on one side of the permanent structure, additional channel length will be necessary to connect the main portion of the diversion channel to the natural stream channel.

Based on inspection of the site layout, it is determined that these transitions should be 30 feet long at each end of the channel, and that they should have a radius of curvature of 25 feet. The riprap transition lining should extend past the ends of the curved transitions. Since the radius of curvature for these transitions is more than three times the bottom width of the channel, the downstream riprap should extend one and a half times the channel bottom width, or nine feet, from the end of the diversion channel. Thus, according to the specifications in TDOT Section 5.05.7, the minimum downstream length of riprap should be 39 feet. V_P, the velocity at which water is discharged from the pipe, was found to be 6.43 feet per second (using methodologies discussed in Section 4.3.4); thus, machined riprap Class B is required for this transition (Table 3.3.2.1-A). An energy dissipator (Section 4.3.4) is required at the outlet.

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TDOT. *Drainage Manual Ch10*.

3.3.2.5 Temporary Construction Ford





Source: TNWRRC

Definition and Purpose

A temporary construction ford is a short-term erosion control measure designed to facilitate the movement of construction vehicles and equipment across ditches, swales, or shallow watercourses without causing damage to the channel and increasing sediment transport. Typically, it consists of a layer of stone placed over geotextile fabric to create a stable crossing while minimizing erosion and sediment disturbance. This method ensures that intermittent flows are not obstructed or contaminated while providing a durable access point for construction activities.

Appropriate Applications

Temporary construction fords are ideal for short-term crossings where wide, shallow depressions, side ditches, or intermittent swales must be traversed during construction. They are most applicable in areas where alternative access routes are impractical, construction activities are expected to last less than a year, crossings are relatively infrequent, and channel banks do not exceed five feet in height (TDOT). These fords provide a stable and efficient solution for moving light-duty construction vehicles across drainage features. Crossing with heavy-duty equipment may be appropriate, depending on the size of the stream.

Limitations and Maintenance

Temporary construction fords must be designed to allow unimpeded water flow, minimizing the risk of upstream ponding. It is best if the accumulation of water does not exceed 12 inches or half the height of existing banks, whichever is smaller (TDOT). Regular inspections, particularly after runoff-producing events, are essential to identify and address blockages, erosion, riprap displacement, or other structural issues. Further, inspections after runoff-producing events are critical to ensure that upgradient EPSC measures are effective in keeping sediment away from the ford. Prompt repairs are necessary to prevent further degradation and full effectiveness of the ford.

Because construction fords are not suitable for use in larger streams, wetlands, jurisdictional water bodies, or water bodies with ecological significance, it is unlikely an ARAP will be required. Additionally, other permits, such as a Section 404 permit from USACE or a TVA 26a permit, are likely not necessary. However, it is best practice to refer to the CGP to understand if the watercourse meets the criteria of a stream. Any additional permits and their conditions are to be considered during the planning stages of the EPSC plans. When crossing larger streams, wetlands, jurisdictional water bodies, or water bodies with ecological significance is necessary, consider alternative crossing methods in Section 3.3.2 and the necessary permits.

Planning and Design Considerations

When planning and designing a temporary construction ford, it is best for the construction to take during periods of dry conditions to minimize disturbance. The ford is to be positioned perpendicular to the watercourse whenever possible, with any necessary skew limited to 15 degrees. The location chosen for crossing is to minimize disruption to the channel banks, bottom, and riparian area. To ensure stability and accessibility, the approach slopes are not to exceed 5H:1V, and any excavated materials are to be removed by pulling it back rather than pushing it into the watercourse. The crossing width ranges between 12 and 20 feet, with side slopes no steeper than 2H:1V (TDOT). When constructing, a stream diversion can be used in order to install the crossing, and again when the ford needs to be removed after construction at the site is completed.

Erosion control measures are to be incorporated to prevent surface runoff from construction roads from entering the watercourse, such as low berms or diversions placed along the channel. The ford itself consists of a 12-inch layer or less of Class A-3 conventional riprap placed over Type III geotextile fabric to provide stability. The effect of ponding due to the placement of rock should be considered when proposing this temporary measure, and the depth of the rock should be minimized to the maximum extent possible for there to be a stable base and yet minimizing impacts to the waters.

Once the ford is no longer needed, all materials will be removed, and the channel banks restored to their original dimensions. Exposed soil must be stabilized with appropriate erosion control methods, such as seeding with mulch or erosion control blankets. Properly designed and installed temporary fords allow for safe, short-term vehicle crossings while minimizing sedimentation and hydrologic impacts.

Example Application

-Example courtesy of TDOT-

Given:

A temporary construction ford is to be constructed across a roadway side ditch. The ditch has a bottom width of 4 feet, side slopes are 3.5 feet high at slopes of 3H:1V, and it is grass-lined.

Determine:

The required quantities and dimensions for the crossing include:

a.) Length of the excavation along the bank
b.) Volume of the excavation along the bank
c.) Volume to be excavated on the ditch bottom
d.) Total excavation volume
e.) Area and volume to be covered by riprap as well as the total amount of conventional riprap.

Solution:

a.) Since the ditch has a grassed lining, the bottom will be excavated to a depth of 12 inches in order to provide a firm base for the stone layer and to ensure that it will not cause an obstruction to flow. Thus, the total height from the bottom of the excavation to the point where the excavated slope meets the existing grade will be 4.5 feet. Since the approaches will be excavated to a 5H:1V slope, the length of the excavation on one side will be:

$$L = H \times Slope = 4.5 \times 5 = 22.5 \text{ ft; use } 23 \text{ ft}$$

b.) The volume within the excavated area on one bank will be equal to the length of the excavation times the average cross-sectional area of the excavation. Since the proposed ford will have a width of 12 feet, and the side slopes of the excavation will be 2H:1V, the cross-sectional area adjacent to the channel may be computed as a trapezoid:

$$Area_{channel} = b \times H + Slope \times H^2 = 12 \times 4.5 + 2 \times 4.5^2 = 94.5 \text{ ft}^2$$

Since the cross-sectional area is zero where the excavated slope meets the existing grade, the average cross-sectional area may be computed as half of the

area adjacent to the channel, or 47.3 SF. Thus, the total volume of excavation for one side of the channel is:

$$V = L \times A_{avg} = 23 \times 47.3 = 1087.9 \text{ ft}^3$$

However, a large portion of this volume is "empty air" above the existing ditch side slope. As stated above, the height of the channel is 3.5 feet. Thus, the length along the 3H:1V slope is:

$$L = H \times \text{Slope} = 3.5 \times 3 = 10.5 \text{ ft}$$

Using similar logic as was employed above, the volume of "empty air" above the channel side slope may be computed as:

$$\text{Air}_{empty} = ½ \times (12 \times 3.5 + 2 \times 3.5^2) \times 10.5 = 349.1 \text{ ft}^3$$

The net volume to be excavated on one side of the channel may then be computed as:

$$V_{side} = V - \text{Air}_{empty} = 1087.9 - 349.1 = 738.8 \text{ ft}^3$$

c.) The volume to be excavated on the ditch bottom may be computed as:

$$V_{bottom} = H \times b \times W = 1 \times 12 \times 4 = 48 \text{ ft}^3$$

d.) The total volume to be excavated is thus:

$$V_{total} = V_{side.1} + V_{side.2} + V_{bottom} = 738.8 + 738.8 + 48 = 1525.6 \text{ ft}^3 = 56.5 \text{ yard}^3$$

e.) The total length of the ford is 50 feet (23 feet on each side plus the 4-foot width of the ditch). Since the width of the ford will be 12 feet, the area covered by riprap will be:

$$A_{riprap} = W \times L_{total} = 12 \times 50 = 600 \text{ ft}^2$$

Since the depth of the riprap will be 1 foot, the volume will be 600 ft^3, which is equal to 22.2 yard^3. If the weight of riprap is 1.75 tons per cubic yard, the final quantity of riprap is 38.9 tons.

References

TDOT. *Drainage Manual Ch10*.

3.3.2.6 Temporary Stream Crossing





Source: TDEC

Definition and Purpose

A temporary stream crossing may consist of pipes or culverts placed within a flowing stream or watercourse and covered with clean stone or riprap to provide site access for construction vehicles. It can also be a temporary bridge made of wood, steel, or other materials. As the most common type of stream crossing, culverts are cost-effective, readily available, and capable of supporting heavy equipment. Bridges, while less common due to the time and cost to construct, as less intrusive to the stream. Properly designed stream crossings reduce damage to the stream bed, channel, or banks during use; however, they may contribute to erosion during installation and removal.

Appropriate Applications

Temporary stream crossings can be installed where construction traffic cannot be routed around a stream. Culvert crossings are an ideal measure for crossings with heavy machinery, where many crossings occur throughout the workday, and when the channel is too wide for normal bridge construction (VDEQ, 2024). Bridge crossings are ideal in sensitive waterways as bridge construction results in less disturbance to the streambed and banks, as well as the least impediments to aquatic organism passage and flow compared to culverts. Stream crossings may also be used as access for structures in need of repair or that are proposed for repair, such as bridges.

Limitations and Maintenance

Temporary crossing structures are generally used for projects lasting less than one year. Crossings are to be removed once no longer needed. Storms larger than the design event may yield significant damage to the crossing, such as washouts, blockages, overtopping, or loss of materials. For projects longer than two years, a larger design storm may be needed in order to prevent washout of the crossing. If a storm occurs that is beyond the design storm and washes the crossing out, it is the permittee's responsibility to clean up the material, rebuild the crossing, and consider a larger design storm capacity. Larger streams may not be suitable to have temporary culvert crossings due to the amount of drainage area and flow that is draining to that point.

Inspect the crossing after every rainfall and at least twice a week during required inspections. Repair any damage to the crossing as soon as possible. Unstable stream banks may need lining with conventional riprap or appropriate stabilization measures. If the crossing detains and traps sediment, the crossing may require total replacement. It may not be sufficient to clean and properly dispose of the sediment, as the crossing will likely continue to trap sediment.

Installing temporary culvert crossings in jurisdictional waters requires additional permits, such as an ARAP, and therefore, both the conditions of the CGP permit and ARAP must be followed. A Section 404 permit from USACE may also be required. If the proposed crossing is to be completed in a TVA reservoir, a TVA 26a permit may also be necessary. Local permitting may be required in addition to state and federal permitting requirements. Consider the criteria and conditions of the necessary permits during the planning stages of the project and EPSC plans. Temporary culvert(s) may not be permitted in streams with sensitive or endangered aquatic life. Culverts may create hydraulic jumps or fish barriers, thereby preventing passage (see example photograph) if they are not installed correctly. Temporary culverts should only be constructed when necessary, as they create the greatest obstructions to flood flow and aquatic habitat (GSWCC, 2016).


Source: GSWCC (2016)

Planning and Design Considerations

The culverts used for the temporary culvert crossing can vary in shape, size, and material (GSWCC, 2016; VDEQ, 2024). The design of the temporary culvert crossing requires hydrologic and hydraulic analysis to ensure it can safely convey expected flow rates and velocities. This is necessary to compute the ideal number, diameter, and length of pipe.

Include all design specifications in the EPSC plans. Within the ARAP General Permit, there are requirements to embed the culvert in the streambed. Placing the culvert inverts at the same elevation as the streambed is the least impactful option for a temporary crossing. This allows the substrate not to be disturbed and to remain at the same elevation before and after construction.

There are various planning and design considerations to be aware of before constructing temporary culvert crossings. The crossings need to be protected from washout during periods of high flows. This can be achieved by diverting high flows around or over the structures. To limit disturbances and preserve the local ecosystem to the maximum extent possible, implement the following (TDOT):

- Limit the clearing or riparian vegetation as much as possible;
- During installation, divert the flowing water around the structure;
- Design the crossing perpendicular to avoid impeding stream flow;
- Design the crossing to support the maximum anticipated loads and width of equipment such that maintenance burdens are not repeatedly observed;
- Place culverts on the streambed to minimize fish blockages and hydraulic jumps;
- Extend the culverts at least one foot beyond the upstream and downstream toe of placed aggregate; and
- Inspect channel conditions to see if additional or other stream stabilization measures may be necessary.

The culverts need to be sized to convey the peak flow from the designated design storm: either the 2-year, 24-hour, or 5-year, 24-hour, depending on the quality of the stream. Check local regulations if a larger design storm is required. The minimum pipe size suggested is 18 inches (TDOT). The diameter of *circular pipes* can be computed following the reformulated version of Manning's Equation (Eqn 15, Section 3.3.2.4). A single culvert may result in a pipe diameter too large for the channel dimensions. In such cases, Table 3.3.2.6-A provides the equivalency of pipe capacities. For example, a 36-inch pipe has the capacity of three 24-inch pipes. When multiple culverts are desired, they should be separated by one-half the diameter of the culvert or 12 inches, whichever distance is greater.

Table 3.3.2.6-A: Number of smaller diameter culverts (top row) required to achieve the same flow capacities as larger diameter culverts (first column). Source: Wisconsin Tubing.

Diameter (in)	18	24	30	36	42	48	54
18	1						
24	2	1					
30	3	2	1				
36	5	3	2	1			
42	7	4	3	2	1		
48	10	5	3	2	2	1	
54	13	7	4	3	2	2	1
60	16	8	5	4	3	2	2
66	20	10	6	4	3	2	2
72	25	12	8	5	4	3	2
84	35	18	11	7	5	4	3

Before placement of the pipe culvert(s) and aggregate, place a geotextile fabric on the stream-bed and banks to prevent subgrade sediment migration. Ensure the geotextile covers the streambed and extends a minimum of six inches and a maximum of one foot beyond the end of the culvert and bedding material. Cover the culvert(s) with clean small riprap, such as Class A-1, such that the depth of conventional riprap above the top of the culvert is one-half the diameter of the culvert or 18 inches, whichever is greater. Use a geotextile fabric to separate the Class A-1 riprap from No. 57 stone, which is to be used as the top six inches of the crossing. For sites that are crossing an ETW or waters with unavailable parameters for siltation, use a nine-inch layer of Class A-3 riprap instead of the six inches of 57 stone. Crown the top of the culvert crossing in the center of the crossing above the channel banks and grade down to each approach. Extend the conventional riprap past the top of the bank of each approach to prevent potential erosion. Ensure the top width of the crossing does not exceed 20 feet.


Source: TDEC

A provision to prevent construction road runoff from entering the stream is necessary. The preferred method for accomplishing this is to provide low approaches that will form “sag” points on either side of the stream channel (TDOT). Direct runoff into these “sag” points in erosion-resistant areas adjacent to the access road. These low approaches will also facilitate the safe passage of flood flows greater than the design flow rate. If possible, construct the “sag” points such that they are no lower than the crown of the temporary culvert. Where “sag” points cannot be constructed, low berms, six inches high with 5H:1V side slopes, may be placed on either side of the channel to divert flows.


Source: TDEC

The culverts, rock, and geotextile should be removed immediately after construction is finished, and the stream-bed and banks must be stabilized and restored to pre-construction conditions.

Bridges are to be constructed with appropriate stringer and decking materials, such as logs, sawn timber, concrete, or metal, that can safely support anticipated loads. If installing the bridge requires work within the channel, a temporary stream diversion must be implemented, such that construction can be completed in dry working conditions to prevent sedimentation and protect water quality. The bridge should be built at or above the bank elevation to avoid trapping floating debris, with abutments placed on stable, parallel banks. The structure needs to span the entire channel, and additional piers or supports may only be used when the stream exceeds eight feet in width; streams narrower than eight feet should not need to contain in-channel supports. This may require additional permits. Decking and any optional run planks or curbs should be securely fastened to ensure safe equipment passage and to minimize the risk of soil being dislodged into the stream.

Once installed, the bridge needs to be anchored at one end so that it can lift freely in high flows without obstructing the channel, using large trees, boulders, or driven steel anchors as secure tie-points. All disturbed areas should be stabilized, and temporary bridges located on public rights-of-way may need additional permissions. All bridges should be designed by a qualified engineer to ensure adequate structural capacity.

Example Application

-Example courtesy of TDOT-

Given:

A temporary culvert crossing is to be constructed across a stream in Wilson County, TN (Type II rainfall distribution), which has the following characteristics:

- Drainage Area = 120 acres (0.1875 square miles)
- Slope of stream at site = 2.0%
- Channel bottom width = 8 feet
- The channel side slopes are 3.5 feet high at a slope of 1.5H:1V
- Runoff Curve Number CN=75 Wilson County, TN
- Time of Concentration, T_c =1 hour
- Assume no pond and swamp areas, F_p =1.

Determine:

The temporary culvert crossing design for this site and compute the required quantities.

Solution:

Step 1- Determine the design flow rate: The design flow rate should be based on the 2-Year, 24-hour storm. The Rational method may be applicable to estimate the design flow rate based on professional judgement; however, the NRCS method is more appropriate. For a 2-year, 24-hour storm, the rainfall depth, P = 3.64 in. Compute the various variables required in Eqn 10 from Section 2.1.3.

a.) Runoff depth, Q_{CN}, Eqns 1-3 from Section 2.1.1:

$$S = 1000 / CN - 10 = 3.33$$

$$I_a = \lambda \times S; \text{ assume } \lambda = 0.2$$

$$I_a = 0.2 \times 3.33 = 0.667 \text{ in}$$

$$Q_{CN} = \frac{(P - I_s)^2}{P - I_a + S} = \frac{(3.64 - 0.667)^2}{3.64 - 0.667 + 3.33} = 1.4 \text{ in.}$$

b.) Unit peak discharge, q_u:

$$I_a / P = 0.667 / 3.64 = 0.183$$

q_u = 340 csm/in (Exhibit 4-II or 5-II, NRCS, 1986)

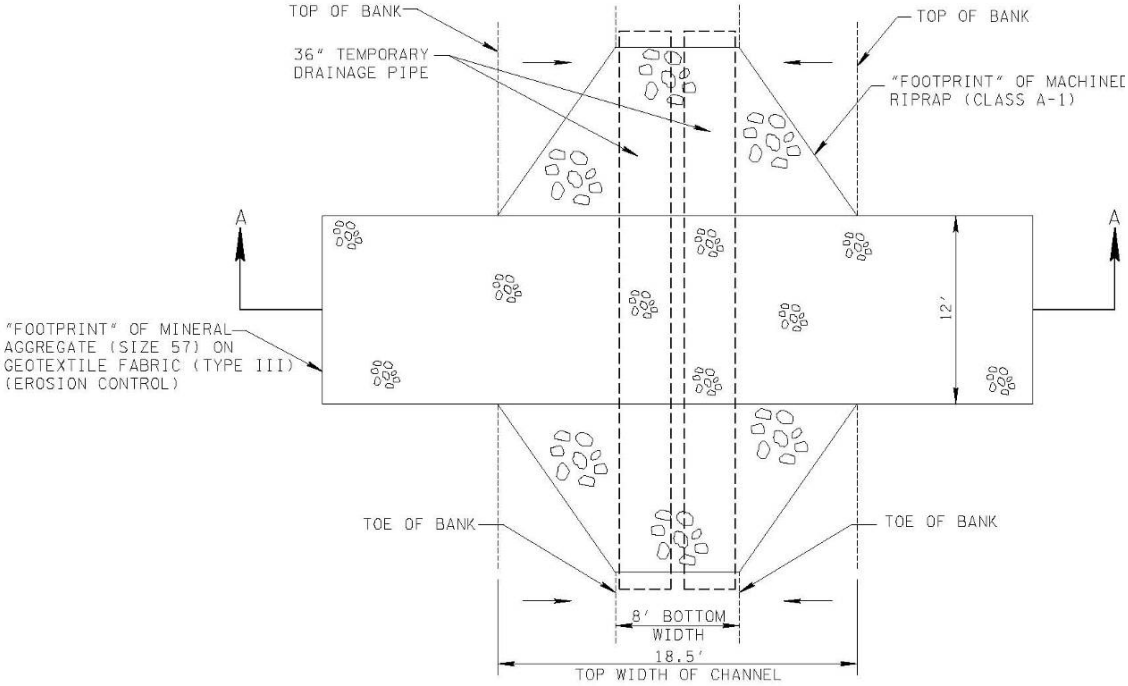
c.) Peak discharge, q_p:

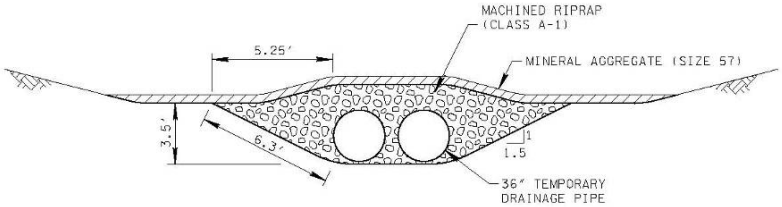
$$q_p = q_u \times A_m \times CN \times F_p = 340 \times 0.1875 \times 1.4 \times 1 = 89.3 \text{ ft}^3/\text{s}$$

Step 2- Select pipe size and number: Using Eqn 15 (Section 3.3.2.4), the diameter pipe (assuming corrugated metal; n = 0.022) required to convey 89.3 cubic feet per second can be calculated:

$$D = 16 \times \left(\frac{q_p \times n}{\sqrt{S}}\right)^{3/8} = 16 \times \left(\frac{89.3 \times 0.022}{\sqrt{0.02}}\right)^{3/8} = 42.9 \text{ inches}$$

Thus, the smallest diameter pipe that can convey 89.3 cubic feet per second of water is 48 inches. However, as indicated on the standard drawing, this pipe would require two feet of cover in addition to six inches of mineral aggregate (size 57). Thus, the total height of this structure would be 6.5 feet, which is judged to be too large to fit into the channel height. As an alternative, twin 36-inch pipes are considered to handle the same flow rate (Table 3.3.2.6-A). The spacing between the pipes should be 1.5 feet for a total width of 7.5 feet, which can be accommodated in the channel bottom. Further, these pipes require 1.5 feet of cover, which, combined with 6 inches of mineral aggregate (size 57), results in a total height of 5.0 feet. This height is judged to be a much better fit compared to the channel height, and twin 36-inch pipes are selected for the crossing. An example drawing of the design is presented.





SECTION A-A

References

GSWCC. (2016). *Manual for Erosion and Sediment Control in Georgia*.
TDOT. *Drainage Manual Ch10*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*
Wisconsin Tubing. *Flow Capacity of Multiple Culvert Installation*.

3.3.2.7 Utility Stream Crossing



Source: VDEQ (2024)

Definition and Purpose
The utility stream crossing measure is to help protect water bodies from disturbed sediment during the construction of a utility line. Crossings entail temporarily diverting the stream from its normal flow course or can employ trenchless technologies like horizontal directional drilling.

Appropriate Applications
This is applicable where utility lines are crossing a stream, wetland, or other jurisdictional waterbody. These can be fiber, electric, water, sewer, storm sewer, or other utility pipes or conduits that are to be buried and pass under a waterbody.

Limitations and Maintenance
Work completed around streams should be the least impactful alternative; often, this determines the construction method. The least impactful method discussed in this section is horizontal directional drilling. It provides a utility crossing of a surface water and does not physically touch the stream subgrade with construction equipment.

Care must be taken to inspect any stream crossing area at the end of each day to confirm that construction materials cannot be moved downstream. Proper maintenance and inspection for each type of flow diversion measure ensures that best practices are followed, and the working area remains dry.

Utility crossings in jurisdictional waters require additional permits, such as an ARAP, and therefore, both the conditions of the CGP and ARAP must be followed. A Section 404 permit from USACE may also be required. If the proposed crossing is to be completed in a TVA reservoir, a TVA 26a permit may also be necessary. Local permitting may be required in addition to state and federal permitting requirements. Consider the criteria and conditions of the necessary permits during the planning stages of the project and EPSC plans.

Planning and Design Considerations

Utility construction frequently crosses and impacts streams. Thus, there is potential for excessive sediment loss by both the disturbance of the approach areas and by the work within the stream. To the greatest extent possible, utility line crossings are to be perpendicular to the stream so that they overlap the streambed the least amount possible (VDEQ, 2024).

In cases where instream work is unavoidable, it is necessary to provide adequate protection for sediment loss while minimizing the amount of encroachment and time spent working in the channel. If the utility crossing is through an ephemeral or intermittent stream and can be completed within a few days, coordinate the disturbance of the stream and construction during dry weather. If that is not possible, other measures can be taken to minimize siltation in the stream (WVDEP, 2016).



Selecting the appropriate method for a utility stream crossing can be challenging as it requires balancing environmental impacts, feasibility, and regulatory compliance. Horizontal directional drilling and Jack and Bore drilling are preferred techniques because they minimize disturbance to the waterbody by avoiding direct excavation of the streambed. These trenchless methods are particularly advantageous in environmentally sensitive areas, as they reduce sedimentation, protect aquatic habitats, and maintain natural stream flow (VDEQ, 2024; WVDEP, 2016). Horizontal direction drilling is ideal for long-distance crossings and challenging terrain, providing a cost-effective solution with minimal surface disruption. Jack and Bore is well-suited for shorter, more controlled crossings, especially in areas with high groundwater or unstable soils. Given the complexity of selecting the right approach, designers and plan reviewers may consider conducting on-site evaluations to determine the most suitable method for each project. When neither option is feasible, open trench methods exist, which are more intrusive and likely time-consuming.

Horizontal directional drilling is a trenchless construction method used to install underground pipelines, conduits, and cables with minimal surface disruption. Position entry and exit points at least 50 feet from the top of the stream bank, allowing the drill to achieve sufficient depth beneath the streambed and minimizing the risk of disturbing the stream environment. The bore depth must be adequate to prevent the inadvertent release of drilling fluids, which could contaminate the watercourse. Additionally, develop a site specific contingency and containment plan to address any unintended release of drilling fluids, ensuring that it is readily available on-site during construction for immediate response. Furthermore, conducting thorough geotechnical investigations is crucial to understanding soil composition and subsurface conditions. This information aids in designing an appropriate bore path and selecting suitable drilling techniques, reducing the likelihood of complications during operations.

Jack and Bore drilling, also known as auger boring, is a trenchless construction method used to install underground pipes, casings, or conduits beneath the stream. A bore pit is excavated at the entry point, and a receiving pit is established at the exit point, both of which are to be located at least 20 feet from the top of the stream bank. In this method, a steel casing is placed at the entry pit, and a hydraulic jack pushes the casing forward while an auger drill inside the casing removes the soil. The auger rotates and removes soil from the front of the bore as the casing is jacked forward, gradually advancing through the earth. Once the casing reaches the receiving pit, the auger is removed, and the utility pipe is installed inside the casing. Jack and Bore drilling is not ideal in loose or wet soils or where open space is limited.

An open trench requires clearing, excavation, pipe installation, riprap backfill, and bank restoration. This results in removing trees, vegetation, and root systems that hold stream banks together and could potentially cause additional bank stabilization issues.


Source: TDEC

a. *Stream Diversion Channel*: Consider this method if construction will remain in the area of the stream for an extended period (longer than 72 hours) and if site conditions are agreeable. See Section 3.3.2.4 for guidance on stream diversion channels. See below for notes that apply specifically to utility crossings.
    - This method allows continuous base flow of the stream around the construction of the utility crossing. Construct the utility trench perpendicular to the stream bank and fully cross the stream bed and bank prior to removing the diversion channel; and

- Once the trench and surrounding area have been backfilled and stabilized, the stream can be rediverted back to the original streambed alignment. Stabilize the disturbed stream-bed and banks and the approach areas immediately following the attainment of the final grade.

b. *Instream Diversion*: This can be used when the stream is wide enough (10 feet or wider). See Section 3.3.2.3 for guidance on stream diversion channels. See below for notes that apply specifically to utility crossings.

- Remove large rocks, woody vegetation, or other material from the streambed and banks that may get in the way of placing the riprap, sandbags, sheet metal, or wood planks used for diversion or installing the utility pipe or line;
- When the stream flow is successfully diverted, dewater the work area;
- Excavate the utility trench in the work area and install the utility line in half of the streambed;
- Stabilize the stream-bed and banks and the approach areas for the initial half of the installed utility line immediately following the attainment of the final grade; and
- Shift the flow from one side of the diversion to the other, such that the next half of the utility crossing can be constructed, and the water can be diverted to the newly stabilized portion of the stream.

c. *Bypass Pumping*: Bypass pumping is beneficial for work that is to be completed within a week and for smaller streams with low baseflow. This is due to a person having to run the pump and be present to manually turn it on and off. See Section 3.3.2.4 for details on bypass pumping. See below for notes that apply specifically to utility crossings.

- This method allows for the use of a temporary pump to transfer water from upstream to downstream and for placing berms across the streambed to prevent flow from entering the work area; and
- If stormwater does enter the work area, it will need to be dewatered. This allows for work to be done in the dry and as well as allowing the streambed to be stabilized before reintroducing the stream's flow back into the channel.

There are a plethora of good practices when installing utility crossings.

- As with all utility line crossings, approach areas may need to be controlled with perimeter measures such as silt fence;
- For a sewer line crossing a stream, consider providing non-erodible fill and cover, such as concrete or controlled low-strength materials (flowable fill), and trench plugs at each end of the crossing;

- Avoid placing manholes near waterbodies and consider providing a minimum distance of 50 feet from the stream bank to the nearest edge of the manhole. Default to what is allowable per the ARAP;
- It is important that the utility crossing prevent permanent impoundment or loss of normal or baseflow;
- Dewatering of the utility trench and construction area may be necessary during construction. A good practice to use in this case would be to pump to a sediment filter bag that is located downstream of the work area. See Section 4.4.12.5 for guidance on the use of sediment filter bags;
- It is best not to blast for the excavation of trenches that are parallel or lie within 50 feet of a stream or wetland. This can impact the streambed and cause the loss of baseflow. Blasting may not be permitted per the ARAP;
- It is best practice to have a minimum of one trench plug between manholes and one trench plug at each end of the stream crossing or wetland; and
- It is important to be aware of existing erosion occurring near the proposed utility crossing. Actively incising streambanks requires careful consideration and planning. If the headcut is happening downstream of the utility crossing, there is potential for the erosion to continue upstream, where the utility crossing is proposed. This may encourage the designer to try to pick a more stable portion of the stream for the crossing.

Example Application

For sizing, refer to Section 3.3.2.3. No formal design or quantities are required for trenchless technologies.

References

VDEQ. (2024). *Virginia Stormwater Management Handbook.*
WVDEP. (2016). *Erosion and Sediment Control Best Management Practice Manual*.

3.3.3 Protected Areas

Prior to and during construction, it is crucial to have a visual delineation around natural areas and features such as streams, wetlands, ponds, cemeteries, historically significant sites, sinkholes, etc. Visual demarcation helps to prevent environmental or historical damage as well as to preserve ecosystems and areas of concern. Section 3.3.3 is specifically in reference to non-permitted impacts to waterbodies and other natural features; permitted impacts to waterbodies should be covered by an ARAP or other necessary permit (Section 1.3.2). These natural areas can play a vital role in filtering pollutants, reducing erosion, and maintaining water quality. By safeguarding them, construction activities can minimize ecological degradation and maintain the integrity of the surrounding ecosystem, which is more cost effective than replacing or repairing the areas if destroyed or degraded.

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3.3.3.1 High-Visibility Fencing



Source: TNWRRC

Definition and Purpose
High-visibility fencing is temporary fencing that can be installed to safeguard protected areas. The fencing is intended to restrict clearing and prevent disturbances to sensitive areas and their buffers.

Appropriate Applications
High-visibility fencing may be used to establish clearing limits, restrict construction traffic to designated entrances or roads, and protect areas where marking with survey tape or flagging may not provide adequate protection or visibility. Fencing may be used at the boundary of sensitive areas, their buffers, areas required to be left uncleared, and as necessary to control vehicle access to and on the site.

Limitations and Maintenance
Repair or replace the fencing if it has been damaged or if the visibility has been reduced.

Planning and Design Criteria
Ensure high-visibility plastic fencing is composed of a high-density polyethylene material, colored orange, four feet in height, and has a tensile strength of 360 pounds per foot using the ASTM D4595 testing method. Posts for the fencing can be steel or wood and placed every six feet on center (maximum) or as needed to ensure rigidity. Fasten the fencing material to the post every six inches with a polyethylene tie. On long, continuous lengths of

fencing, a tension wire or rope may be used as a top stringer to prevent sagging between posts. Fencing should not be wired or stapled to trees.

If appropriate, a fabric silt fence can be used in accordance with silt fencing requirements (refer to Section 4.4.10) to act as a high-visibility fence. The silt fence is to be three feet high and highly visible to meet the requirement of high visibility fencing.

Metal fencing is also permitted and is to be installed according to manufacturer's specifications. Metal fences should be at least three feet high and highly visible to meet the requirements of high visibility fencing.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

None.

3.3.3.2 Riparian Buffer Zones



Source: TDEC

Definition and Purpose

A riparian buffer zone is a protected area along an onsite water resource that can be temporary or permanent. Its primary function is to physically protect the water resource and act as a separator between the water body and disturbance. Though not a sediment control, riparian buffers provide water quality benefits through filtration by the vegetation. These zones also protect banks and promote bank stabilization by slowing construction site runoff with vegetative resistance (City of Albany, 2024; NCDEQ, 2013).

Appropriate Applications

All streams, wetlands, ponds, and reservoirs are required to have riparian buffer zones between the top of the bank and soil disturbance during construction and in regulated municipal areas after construction. However, buffers can be beneficial in additional areas, such as ditches or wet weather conveyances.

Several municipalities and counties have more restrictive requirements for the width and/or maintenance of riparian buffer zones. While the CGP only has requirements for temporary construction buffers, MS4s may have permanent buffer requirements that must also be met. If there are differences between local and CGP buffer requirements, the more conservative of the two must be met.

Limitations and Maintenance



Source: TDEC

Inspections must focus on the stability of the riparian buffer zone, examining the zone for transported sediment and erosion, and ensuring buffer boundaries are well defined and clearly marked in the field. If erosion or sediment deposition is found within the buffer zone, implement measures to minimize upgradient erosion and restore the buffer. Disturbances to riparian buffers are not permitted unless the designer justifies encroachment, which typically occurs on roadway or utility line projects. In the case where the temporary average buffer width requirement cannot be met, designers must implement other measures to obtain riparian buffer equivalency.

Maintenance of riparian buffer zones may extend into post-construction phases. This depends on the location of the project: if it is within the limits of a regulated municipality, permanent buffers may be required with a continuous maintenance agreement. Permanent buffer widths can vary between municipalities and CGP requirements. This information can be important in the construction planning process, and owners are encouraged to reach out to the local MS4 for pre- and post-construction requirements.

Planning and Design Considerations

Effective riparian buffer zones consist of an undisturbed tree line along the waterbody and upland vegetation. The CGP requires preserved buffers to have an average width of 30 feet between the waterbody and limits of disturbance, with a minimum of 15 feet at any given location. Waters with unavailable parameters for siltation and ETWs are required to have 60 feet of undisturbed vegetation on average, with a minimum width of 30 feet. Though there is no one ideal buffer zone width, the width is directly related to its effectiveness (ALSWCC, 2018). For optimal stormwater treatment, it is recommended to convert concentrated flow into sheet flow with a level spreader or similar prior to discharging into the riparian buffer zone to prevent erosion within the buffer zone. If a buffer is disturbed, promptly stabilize disturbed buffers with a dense cover of strong-rooted grasses, native plants, and native trees.

All construction-related materials and equipment must be stored outside the buffer area. Measures such as high-visibility fencing (Section 3.3.3.1) are ideal to install along the edge of the riparian buffer zone, creating a visual and physical barrier that prevents inadvertent disturbance. Ensure that EPSC measures are installed upgradient from the buffer to protect it from sediment-laden runoff.

An example of a riparian buffer zone plan is as follows:

1. All areas of the riparian buffer zone undergoing restoration must be planted with non-invasive or native vegetation and nurse crops. Mixing cover crops with native seeds may be necessary, depending on the time of year;
2. Refer to Section 4.2.6.11 (Vegetation and Landscaping: Permanent) for guidance on soil bed preparation for native mixes;
3. EPSC measures must be installed to protect the water resource during restoration. These measures can include coir matting, erosion control blankets, wattles, etc., to stabilize the area short-term and long-term. All measures used within the buffer should be made of natural materials that are biodegradable and can be left in place.
4. Plant species, density, placement, and diversity in the buffer restoration plan must be appropriate to ensure a healthy buffer zone. See Appendix D for example seed mixes, planting seasons, and application rates.
5. Native seedlings can take several years to fully establish and may require maintenance, such as the removal of invasive species. Vegetation mortality must be included in planting densities to ensure meeting the 70% density requirement per the CGP. This benchmark may be difficult to obtain by natives alone; therefore, supplementing with turf grasses and nursery crops may be necessary.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

City of Albany. (2024). *Erosion Prevention and Sediment Control Manual*.

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual.*

3.3.3.3 Sensitive Areas



Source: TNWRRC

Definition and Purpose

Sensitive areas require protection from construction activities such as clearing, grading, mowing, staging activities, materials storage, and other related activities. Identify such areas with high visibility markings, flags, or fencing to make them known to all personnel.

Appropriate Applications

Features considered to be sensitive need to be identified. Sensitive areas can include tree preservation areas, streams, wetlands, endangered or protected species and/or habitats, water quality buffers, mitigation or stream relocation boundaries, sinkholes, stormwater treatment areas, caves, cemeteries, and historic preservation areas.

Limitations and Maintenance

Boundary markers or high-visibility fencing must be maintained throughout construction. Markers that have been damaged, removed, or degraded to the point that they are no longer visible require replacement. At the end of construction, remove all markers unless a regulatory authority has required them to be left in place.

Planning and Design Criteria

Any sensitive area within the project boundaries is to be identified in the SWPPP and on the EPSC plans. It is the responsibility of the design professional to clearly label all areas on the plans and specify the type of marking materials to be used. Mark all sensitive features in the field accordingly. If these areas are disturbed, additional site design components may be needed to meet local or state requirements.

Sensitive areas that require markings and protection include:

- Streams and wetland buffers;
- Permitted Impact;
    - If the ARAP allows a specific area of stream or wetland encroachment, clearly marking these boundaries in the field aids in maintaining compliance with the ARAP conditions.
- Stream mitigation or relocation boundaries;
    - It is likely that a two-step field marking process is necessary for stream relocations and/or mitigation. The first boundary should identify the permitted impacts to the natural resource and should occur before work in the area. Once the relocation or mitigation has been installed and stabilized, these areas should be marked to show the outer limits and prevent disturbance or damage to vegetation.
- Sinkholes;
    - Sinkhole basins should be protected from sediment from construction sites by using appropriate erosion prevention and sediment controls upgradient from the basin. Leaving sinkhole basins undisturbed provides additional protection. It should be noted that discharges to sinkholes may require an underground injection control permit (Section 1.3.2).
- Areas to remain undisturbed;
    - Often, undisturbed open space requirements are established and enforced by the local jurisdiction. The amount of undisturbed open space per development is typically a percentage of the development. Disturbing these areas can lead to additional development restrictions or mitigation requirements. In addition, disturbed areas increase the required capacity of sediment control. Disturbing areas of land that were intended to be left undisturbed may result in overwhelming sediment controls and potential violations.
- Threatened and/or endangered species habitat; and
    - Critical habitat for threatened or endangered species should be protected from land-disturbing activities. These areas should be clearly identified on the plans and in the field to prevent inadvertent disturbance or encroachment.
- Stormwater management areas.

- Land-disturbing activities destroy the infiltration capacity of soils by changing the soil structure, compacting the soils, and subjecting soil organic matter to a more rapid decay process. Many stormwater management practices, such as bioretention areas and water quality swales, rely on the soil infiltration capacity. When the infiltration capacity of the in-situ soils is substantially altered, the area may no longer be suitable for permanent stormwater management controls.

Many types of boundary markers are available. Flags, stakes, posts, or fencing can be used as field boundary markers. Any marker must be highly visible and installed along the outer perimeter of the boundaries of the protected feature. Bright colors and highly distinguishable marking materials, such as orange fencing, neon or brightly colored flags, or highly visible signage, are the best for construction. Some markers will be temporary (such as ARAP permit boundaries) while others may be more permanent (such as permanent buffer zones). The decision about the type of marker to be used at the site may in part be dictated by the lifespan of the feature being marked.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

None.

3.3.3.4 Tree Preservation



Source: TDEC

Definition and Purpose

Tree preservation includes practices to protect desirable trees from damage during construction activities. Trees can have present or future value, as healthy, mature trees enhance property values, improve aesthetics, promote soil stability, and filter air pollutants (NCDEQ, 2013; City of Albany, 2024).

Appropriate Applications

These practices can be applied to any construction site with existing trees.

Limitations and Maintenance

In spite of precautions, some damage to protected trees may occur. If damage to existing trees occurs, repair the damage by the following maintenance specifications (NCDEQ, 2013):

- Repair roots by cutting off the damaged areas and painting them with tree paint. Spread peat moss or moist topsoil over any exposed roots;
- Repair damage to bark by trimming around the damaged area, tapering the cut to provide drainage, and painting the area with tree paint; and
- Cut off all damaged tree limbs above the tree collar at the trunk or main branch. Use three separate cuts to avoid peeling bark from healthy areas of the tree.

Planning and Design Criteria

During the site evaluation, note where valuable trees and other natural landscape features are to be preserved, then consider these trees and plants when determining the location of roads, buildings, or other structures. When marking trees to protect, be sure to install practices around the drip line of tree branches as opposed to solely around the trunk (Figure 3.3.3.4-A).

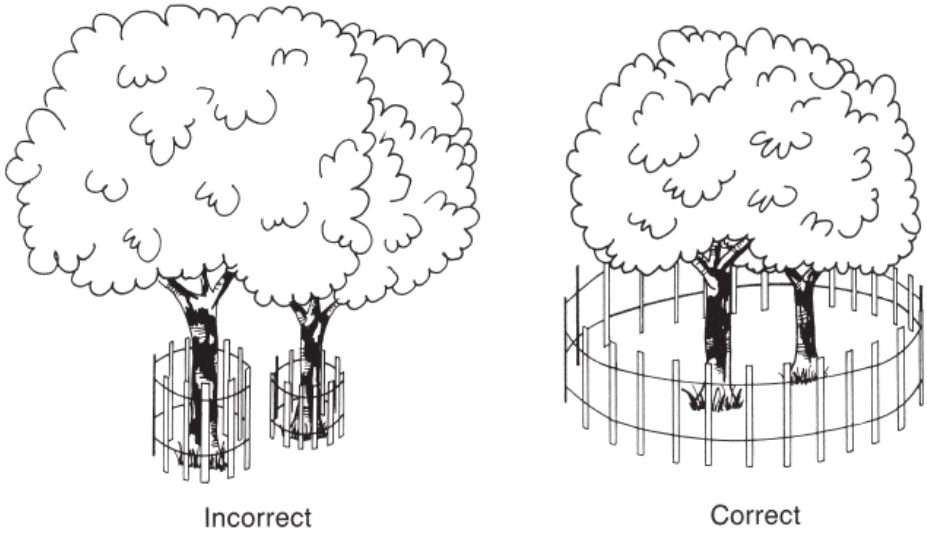


Figure 3.3.3.4-A: Construction barriers should be installed at the drip line of the tree branches. Source: NCDEQ (2013).

Construction activities can significantly injure or kill trees unless protective measures are taken. Trees are considered for preservation for the following benefits:

- They stabilize the soil and prevent erosion;
- They reduce stormwater runoff by intercepting rainfall, promoting infiltration, and lowering the water tables through transpiration;
- They moderate temperature changes, promote shade, and reduce the force of wind;
- They provide buffers and screens against noise and visual disturbances, providing a degree of privacy;
- They filter pollutants from the air and produce oxygen;
- They provide a habitat for animals and birds; and
- They increase property values and improve site aesthetics.

Consider the following characteristics when selecting trees to be preserved and protected (NCDEQ, 2013):

- Tree vigor – Preserve healthy trees. A tree of low vigor is susceptible to damage by environmental changes that occur during site development. Healthy trees are less susceptible to insects and disease. Indications of poor vigor include dead tips of branches, small annual twig growth, stunted leaf size, sparse foliage, and pale foliage. Hollow or rotten trees, cracked, split, or leaning trees, or trees with broken tips have less chance for survival;
- Tree age – Old, picturesque trees may be more aesthetically valuable than smaller, younger trees, but they may require more extensive protection;
- Tree species – Preserve those species that are the most suitable for site conditions and landscape design. Trees that are short-lived, brittle, or are susceptible to attack by insects and disease may be poor choices for preservation;
- Tree aesthetics – Choose trees that are aesthetically pleasing, shapely, large, and colorful. Avoid trees that are leaning and in danger of falling. Occasionally, an odd-shaped tree or one of unusual form may add interest to the landscape if strategically located. However, be sure the tree is healthy; and
- Wildlife benefits – Choose trees that are preferred by wildlife for food, cover, or nesting. A mixture of evergreens and hardwoods may be beneficial. Evergreen trees are important cover during the winter months, whereas hardwoods are more valuable for food.

Although direct contact with equipment is an obvious means of damaging trees, most serious damage is caused by root zone stress from compacting, filling, or excavating within close proximity to trees. Clearly mark boundaries to maintain sufficient undisturbed areas around the trees. The following general criteria should be considered when developing in a wooded area:

- Leave critical areas (such as floodplains, steep slopes, and wetlands) with desirable trees in their natural condition or only partially cleared;
- Locate roadways, storage areas, and parking pads away from valuable tree stands. Follow natural contours, where feasible, to minimize cutting and filling in the vicinity of trees;
- Select trees to be preserved before constructing roads, buildings, and other structures;
- Minimize trenching in areas with trees. Place several utilities in the same trench;
- Designate groups of trees and individual trees to be saved on the EPSC plan sheets and in the SWPPP; and
- Do not excavate, traverse, or fill closer than the drip line or perimeter of the canopy of trees to be preserved.

Use the following guidelines at all times throughout the construction process:

- Do not nail boards to trees during building operations;
- Do not cut tree roots inside the drip line;
- Do not place equipment, construction materials, topsoil, or fill dirt within the limit of the drip line of trees to be preserved;
- If a tree marked for preservation is damaged, remove it and replace it with a tree of the same or similar species, two-inch caliper or larger, and from a balled and burlap nursery stock when activity in the area is complete; and
- During final site cleanup, remove barriers from around trees.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

City of Albany. (2024). *Erosion Prevention and Sediment Control Manual*.
NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.

3.3.4 Construction Exit

A designated construction exit is necessary to minimize the movement of sediment offsite. This specific measure focuses on preventing the spread of sediment by ensuring vehicles leaving the site are not tracking excess mud, dirt, or sediment before entering public roadways. Unlike other EPSC measures that treat runoff, prevent pollution, or stabilize soil, construction exits are unique because they are a distinct component of overall site management and a public safety feature. Large clumps of sediment on the road and dried mud from equipment tires can dislodge at high speeds, impacting other motorists.

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3.3.4 Construction Exit





Source: TDEC

Definition and Purpose

Construction exits often consist of stone pads on a geotextile fabric that are located at any point where traffic will be moving from a construction site onto a public roadway or other paved area. Construction exits may also consist of rumble strips or HDPE material that can be used to reduce track out. The intentions of this measure are to reduce or eliminate the transport of sediment off-site.

Appropriate Applications

This practice is applicable where construction traffic leaves a construction site and enters a public right-of-way.

Reusable HDPE construction exits may be a more economical choice for areas of Tennessee where rock is harder to find and more expensive. Maintenance is required to clean off the construction exit. Manufacturer recommendations should be followed.

Limitations and Maintenance

The exit must be maintained in a condition that will prevent tracking or flow of material onto public rights-of-way or into storm drain systems. This may require periodic top dressing with fresh stone or full replacement of stone and underlying geotextiles as conditions show clogging. All materials spilled, dropped, washed, or tracked from vehicles or site onto roadways or into storm drains must be removed as soon as possible. Additionally, dislodged stones spilling into the adjacent roadway must be cleared as soon as possible.

Planning and Design Criteria

Construction exits are to be installed at any point where construction traffic exits the project. Avoid areas with hydric or saturated soils as well as steep gradients.

A layer of geotextile fabric is used to stabilize and support the aggregate. Extend the geotextile fabric the full length and width of the construction exit and ensure the fabric meets the requirements of the standard specifications for geotextiles, AASHTO designated M-288, erosion control. Construct the stone pad from clean, washed stone. The gradation, width of the stone pad, and length of the pad must be sized appropriately such that tracking onto public roadways is eliminated. There may be specific requirements from local or state requirements for larger projects (e.g., TDOT projects). While no specific dimensions, gradation, or depth of aggregate are required per the CGP, neighboring states recommend minimum lengths of 50 to 70 feet, minimum widths of 12 to 20 feet, minimum depths of six to eight inches, and one to two inch diameter aggregates (TDOT Class A-3 Machined Riprap) (ALSWCC, 2018; GSWCC, 2016; NCDEQ, 2013; VDEQ, 2024). Install, at minimum, a turning radius of 20 feet on each side of the pad where it intersects with the public roadway (TDOT).

Stormwater management around the construction exit must also be considered, such that stormwater cannot flow across the stone pad and into the right-of-way. On sites where the grade toward the public roadway is greater than two percent, a waterbar diversion six to eight inches in depth with 3H:1V side slopes can be constructed at the upper end of the construction exit to prevent stormwater from washing sediment off the construction exit and into the public roadway or storm drain system (GSWCC, 2016).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.
GSWCC. (2016). *Manual for Erosion and Sediment Control in Georgia*.
NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual.*
TDOT*. Drainage Manual Ch10.*
VDEQ (2024). *Virginia Stormwater Management Handbook.*

3.4 Pollution Prevention

Pollution prevention practices are good housekeeping practices that reduce pollution potential. Practices in this category are essential to minimize the release of harmful substances such as trash, debris, construction waste, chemicals, sediments, etc. into the environment. By implementing these measures, construction sites can protect local ecosystems, safeguard water quality, and ensure compliance with environmental regulations.

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3.4.1 Chemical Storage



Source: CalTrans (2017)

Definition and Purpose

Chemical storage areas are equipped to eliminate the discharge of chemicals, fertilizers, and other non-sediment pollutants. This is achieved by minimizing the storage of all such materials onsite, storing onsite materials in a designated area, installing secondary containment, providing overhead or indoor coverage, and conducting regular inspections.

Appropriate Applications

This practice is applicable for the storage of the following materials (CalTrans, 2017):

- Soil stabilizers and binders;
- Pesticides and herbicides;
- Fertilizers;
- Detergents;
- Plaster;
- Petroleum products such as fuel, oil, and grease;
- Asphalt and concrete components;
- Hazardous chemicals such as acids, lime, glues, adhesives, paints, solvents, and curing compounds;
- Concrete compounds; and
- Other materials that may be detrimental if released to the environment.

Limitations and Maintenance

Keep storage areas clean, well-organized, and equipped with ample cleanup supplies as appropriate for the materials being stored. Repair or replace perimeter controls, containment structures, covers, and liners as needed to maintain proper function.

Planning and Design Criteria

Keep an up-to-date inventory of materials stored onsite. All chemicals must be stored in covered areas or in containment systems constructed in or around the storage areas. Further, keep chemicals in their original labeled containers. Reactive, ignitable, or flammable liquids must be stored in compliance with local fire codes. Locate storage areas as far away as possible from waterbodies, sensitive areas, and inlets.

Ideally, store materials that are likely to contribute to pollution indoors. Storage sheds must meet local building and fire code requirements. If indoor storage of materials is not possible, providing coverage is the next best option. Do not store chemicals, drums, or bagged materials directly on the ground. Place these items on a pallet, in secondary containment, and under cover when possible (TDOT, 2017). These methods shield construction materials from raindrop impact and keep them out of flow paths. When providing overhead coverage is not feasible, plastic dome toppers can be purchased and snapped to the top of drums, which prevent rainwater from ponding on the drums.

Ensure all sites have spill kits. If a spill occurs, avoid washing the materials away with water, as the water may run off and spread the contamination. Instead, use dry methods to safely transfer the spilled materials into the designated spill containers for proper handling and disposal. Include emergency contact information for spills, such as the local emergency management agency's contact information. A local TDEC environmental field office (EFO) should be contacted in the event that a chemical spill reaches a waterway. If residual materials remain after cleaning a spill or after construction completion, properly remove materials and any contaminated soil. If the area is to be paved, pave as soon as materials are removed to stabilize the soil and reduce the chance of spreading any contaminants.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

CalTrans. (2017). *Construction Site Best Management Practices (BMP) Manual*.
TDOT. (2017). *EPSC Inspection Manual*.

3.4.2 Concrete Washout



Source: Hamilton Co. (2008)

Definition and Purpose

Concrete washouts are pits lined with plastic (or other watertight materials) or leak-proof containments that can hold and store liquid or slurry concrete washed off heavy machinery and vehicles. The washoff is held in place until it is dried to a point at which it can be disposed of properly.

Appropriate Applications

Concrete washout areas are applicable when:

- Concrete trucks and other concrete-coated equipment are washed onsite;
- Slurries containing portland cement concrete or asphalt concrete are generated, such as from saw cutting, coring, grinding, grooving, and hydro-concrete demolition; and
- Washing of exposed aggregate concrete occurs.

Limitations and Maintenance

Concrete washouts are temporary facilities that require emptying on a routine schedule, such that a minimum freeboard of four inches is provided for above-grade washout facilities and 12 inches for below-grade washout facilities. Remove concrete once it has hardened to ensure sufficient storage for additional concrete washout. Alternatively, remove accumulated materials once 75% of the storage capacity is filled. Inspect plastic linings, geotextiles, and sidewalls of site-built washouts to ensure there are no tears or leaks. During

cleaning processes, pump any excess wash water into a containment vessel that can be properly disposed. Plastic linings may need to be replaced after each cleaning of the washout facility (VDEQ, 2024). Inspect all surfaces of prefabricated washouts to ensure the container is not leaking.

Identify whether washout facilities are being used regularly. If the inspector finds that concrete trucks are being washed out in locations other than designated areas, the inspector must notify the site superintendent and/or operator immediately.

Avoid mixing excess concrete for a pour, and also perform washout of concrete trucks offsite or in designated areas. Do not allow concrete trucks to wash into storm drains, open ditches, streets, or streams. Do not allow excess concrete to be dumped onsite, except in designated areas. Do not wash sweepings from exposed aggregate concrete into the street or storm drainage systems.

Planning and Design Criteria

Educate all construction site personnel on the use of the washout facilities (USEPA, 2012). There are two main types of concrete washouts: prefabricated washout containers and site-built washouts. Private companies often offer heavy-duty, prefabricated concrete washout containers that are delivered to the site. Some services provide only the containers, while others also provide the maintenance and disposal of the materials. When selecting a company to handle concrete waste, ensure they properly dispose of all materials. If the project utilizes a concrete pump truck, the prefabricated container should have an adequate ramp to accommodate the concrete pump truck. When using prefabricated washout containers, ensure containers can withstand heavy impacts and are watertight.



Source: TDEC


Source: TDEC

Site-built washouts can be below-grade in order to prevent breaches. They are constructed by providing a temporary pit or bermed area large enough to handle solids, wash slurry, and rainfall to prevent overflow. They often are 10 ft by 10 ft and include a minimum of 12 inches of freeboard. Above-grade structures may be used if they are sized properly, leak-proof, and maintained regularly to provide a minimum of four inches of freeboard. Site-built concrete washouts may use a 10-mil plastic lining or a nonwoven geotextile lining. When removing concrete and disposing of the hardened concrete, be sure to dispose of the linings separately and not to bury them with the concrete. This is to reduce the amount of microplastics that will be found on-site.

Locate concrete washout areas as far as possible from streams, sinkholes, wetlands, or other sensitive features yet accessible for concrete trucks and waste removal (USEPA, 2012). It is a good practice to use signage to indicate the location of the washout. Line vehicular pathways to and from the washout with appropriately sized gravel to prevent tracking off-site and erosion. Consider having multiple washouts on large sites.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

Hamilton County. (2008). *Stormwater BMP Manual Best Management Practices*.
USEPA. (2012). *Stormwater Best Management Practice: Concrete Washout*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*

3.4.3 Tire Washing



Source: TNWRRC

Definition and Purpose
Tire washing refers to an established station for washing tires and undercarriages prior to construction traffic exiting the construction site. In addition to construction exits, tire washing helps reduce mud, dirt, and sediment tracking onto public roads.

Appropriate Applications
Tire washing facilities are used in combination with construction exits when gravel construction exits do not provide sufficient dirt, mud, and sediment removal from construction equipment. Tire washes are not necessary in all cases but may be ideal for sites located in sensitive areas or where additional sediment control measures are necessary.

Limitations and Maintenance
Remove accumulated sediment in the tire wash rack and surrounding sediment control measures as necessary to maintain system performance. Inspect routinely for damage and repair as needed. This application may require a turnaround or doublewide exit to prevent vehicles from driving through the wash area (CalTrans, 2017).

Using soaps or solvents for vehicle and equipment washing is not covered under the CGP and is prohibited from discharging.

Planning and Design Criteria

A reliable water supply is essential for most tire washing techniques, which can be provided through an overhead tank, pressurized tank, water pipeline, or other means. Wash water must be directed to a sediment trapping device, such as a sediment basin or sediment trap, before being discharged offsite to prevent contamination. Runoff from the wash area may require conveyance to sediment controls or trapping devices via a drainage ditch of appropriate grade, width, and depth. If chlorinated water, such as tap water or hydrant water, is used, runoff should be detained at a minimum of 24 hours before discharge. Pool test kits can be used to test the chlorine content of the water.

There are various methods for tire washing, such as flooded basin, countercurrent channel, low-pressure inundation, and high-pressure cleaning, any of which may be implemented based on which will work best for site-specific or economic constraints.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

CalTrans. (2017). *Construction Site Best Management Practices (BMP) Manual.*
City of Knoxville. (2003). *Best Management Practices Manual*.

3.4.4 Trash and Debris



Source: CalTrans (2017)

Definition and Purpose

The management of trash and debris refers to the proper containment and disposal of macro debris, also known as gross solids. It ensures good housekeeping by providing designated waste collection and scheduling regular disposal. Though not often thought of as a stormwater pollutant, gross solids can leach various chemicals and toxins, yield increased microplastics, and clog drainage infrastructure (Roesner and Kidner, 2007).

Appropriate Applications

Trash and debris management apply to construction sites where waste is generated. These include debris from land clearing, such as trees and shrubs, and rubble from demolished structures; packaging materials, such as wood, paper, and plastic, as well as scrap or surplus building materials like metals, rubber, plastic, and glass; and masonry products. Additionally, construction waste such as brick, mortar, timber, steel, pipe cuttings, electrical scraps, Styrofoam, and other packaging materials requires proper disposal. Lastly, proper disposal of domestic waste, including food containers, beverage cans, plastic wrappers, and cigarettes, and onsite sanitary waste facilities are critical components of this practice.

Limitations and Maintenance

Inspect the site for evidence of trash and construction debris being placed outside of the designated collection areas. Ensure construction waste is collected, removed, and disposed of only at authorized disposal areas. Remove and dispose of macro litter on a regular or as-needed basis and following the completion of construction activities, to prevent container overflow. Spills and overtopping require immediate attention.

To prevent clogging of the storm drainage system, litter and debris removal from drainage grates, trash racks, and ditch lines should be a priority. Inspect trash and debris collection areas before and after wind and/or rain events to ensure that trash and debris are being adequately contained.

Planning and Design Criteria

Effective trash and debris management on construction sites is crucial for reducing environmental impact and maintaining regulatory compliance. Trash and Debris storage areas should be placed to minimize the exposure of building materials, building products, construction wastes, trash, landscape materials, fertilizers, pesticides, herbicides, detergents, sanitary waste, and other materials present on the site to stormwater. Include a designated debris storage area in the SWPPP and onsite drawings when such materials are anticipated. These areas are not to be near sensitive site features like streams, wetlands, sinkholes, flood-prone areas, wet weather conveyances, and drainage infrastructure. It can be helpful to incorporate preventative measures such as berms, dikes, or other means around storage areas to reduce stormwater runoff from contacting waste (CalTrans, 2017). It is best to enclose, cover, or equip all containers with lids to minimize exposure to rain (KYTC, 2015). It is helpful when signage clearly designates waste disposal areas. Collect and place onsite waste in disposal areas before rain events, regardless of if it was sourced from construction activities/personnel or not.

Toxic liquid wastes and chemicals, including oils, solvents, paints, acids, pesticides, and curing compounds, must not be disposed of in construction debris containers and should follow guidance outlined in Section 3.4.1. Construction debris and trash can only be used as fill if explicitly approved by regulatory authorities. Proper planning, consistent maintenance, and adherence to these guidelines ensure effective trash and debris management while protecting water quality and surrounding ecosystems.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

CalTrans. (2017). *Construction Site Best Management Practices (BMP) Manual.*

KTC. (2015). *Kentucky Erosion Prevention and Sediment Control Field Guide.*

Roesner, L. A., & Kidner, E. M. (2007). Improved protocol for the classification and analysis of stormwater-borne solids. In *WEFTEC 2007* (pp. 5539-5566). Water Environment Federation.

3.4.5 Vehicle Maintenance



Source: TDEC

Definition and Purpose

Vehicle maintenance includes procedures and practices to reduce the discharge of pollutants to storm drain systems or to watercourses as a result of vehicle wear and tear. Like other good housekeeping practices, this measure is intended to eliminate non-stormwater discharges and non-sediment pollutants.

Appropriate Applications

Procedures for vehicle maintenance are applicable to projects where heavy equipment and vehicles are maintained on-site.

Limitations and Maintenance

Vehicles and equipment require daily inspections. Leaks should be repaired immediately, and problematic vehicle(s) or equipment should be removed from the project site. Keep ample supplies of spill cleanup materials on-site.

Planning and Design Criteria

Vehicle and equipment maintenance on construction sites can be a significant source of stormwater pollution due to potential spills and leaks of engine fluids, lubricants, and other hazardous materials. While offsite maintenance is preferred (City of Albany, 2024), it is not always practical. When maintenance must occur onsite, proper planning and control measures are necessary to minimize environmental impacts. Confine maintenance activities

to designated areas that are level, located at least 50 feet away from drainage facilities and watercourses, and protected from stormwater run-on and runoff. Additionally, provide cover to maintenance areas where feasible and equip maintenance areas with secondary containment to capture spills and leaks. Use drip pans or absorbent pads under vehicles and equipment during maintenance to catch fluids. For long-term projects, consider constructing permanent or portable covers over maintenance areas. Educate all personnel involved in vehicle and equipment maintenance on proper disposal procedures and spill cleanup protocols.

Develop a comprehensive spill prevention and response plan and provide it to all maintenance personnel. Absorbent spill cleanup materials and spill kits must be readily accessible at maintenance areas, fueling stations, and on fueling trucks. Clean small spills using absorbent materials (i.e., dry methods) instead of hosing down or burying spills. Promptly and properly dispose of all contaminated absorbent materials.

Recycle or properly dispose of all used vehicle fluids, including oil, antifreeze, hydraulic fluids, and cleaning solutions, to prevent contamination of soil and water. Separate waste such as oil filters, batteries, and tires for proper disposal or recycling. Under no circumstances should used oil be placed in dumpsters, poured onto the ground, or into storm drains. Similarly, fuel and lubricant disposal must comply with environmental regulations to prevent contamination.

Signs of leaks, such as soil staining under parked equipment, should be investigated and remediated promptly. Inspect incoming vehicles and equipment, including delivery trucks and subcontractor vehicles, for leaks before being allowed on-site. Vehicles and equipment should be kept clean to prevent excessive buildup of oil and grease, reducing the risk of pollutant runoff.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

CalTans. (2017). *Construction Site Best Management Practices (BMP) Manual*.
City of Albany. (2024). *Erosion Prevention and Sediment Control Manual.*

3.5 Construction Sequencing and Phasing

A description of all construction activities must be documented in the SWPPP and on the EPSC plans. The sequence of all soil disturbing activities are to be included in these documents. Construction sequencing is specific to each project that coordinates the timing of land disturbing activities, installation of EPSC measures, stabilization measures, and permanent stormwater management controls. Though not required, phasing is recommended on all construction sites regardless of size as it can be an effective measure for minimizing disturbance, erosion potential, and sedimentation. Site developers are encouraged to limit the total disturbed area to less than 50 acres at a given time. After a portion of construction is complete and stabilization measures have been implemented, developers can begin construction activities in other onsite locations to maintain a disturbance of less than 50 acres.

While the initial, interim, and final stages of EPSC plans are a form of sequencing, notes detailing sequencing of activities are required. The sequencing notes on the EPSC plans provide a more detailed, order of construction activities and demonstrate compliance with the CGP by detailing that perimeter controls such as silt fence and a construction exit (and/or other necessary EPSC measures) are in place before land disturbing activities occur. When changes in construction activities must occur, amend the sequence schedule in advance to maintain management control.

The following demonstrates an example of basic construction sequencing:

1. Install construction exits, delineate buffers zones, and designate laydown yards;
2. Install temporary EPSC perimeter measures, including sediment traps, sediment basins, silt fence, etc.;
3. Perform onsite clearing and grubbing;
4. Excavate and grade the site;
5. Throughout grading and as drainage patterns change, install appropriate EPSC measures such as, check dams, outlet protection, wattles, silt fence, etc. Continue to maintain existing measures;
6. Implement final stabilization of disturbed soil; and
7. Remove all temporary EPSC measures once permanent stabilization has been established.

If phasing is proposed to minimize disturbance or limit the amount of area disturbed at a given time, notes detailing the phasing are required. For example, when a conveyance runs through a construction site, construction phasing can be used to show how offsite water will remain unpolluted by implementing a diversion or bypass. In this case, specify that a diversion channel will be constructed and stabilized to divert water around the construction.

The diversion reduces the drainage area of the outfall and in turn the volume of stormwater requiring treatment, as detailed in Section 3.3.1. Another example would be placing pipes such that stormwater passes through the construction site without exposure to disturbed soils. When planning to phase the project in order to disturb less than 50 acres at a given time, a construction sequence is required for each phase. While beneficial, sequencing alone does not demonstrate compliance with the CGP. To demonstrate compliance, construction details must be provided in the narrative, notes, and drawings of the EPSC plans and SWPPP. When water features are to be impacted within project limits, notes and sequencing can provide detail to the reviewer that the permitted conditions are acknowledged and met.

The following are generalized examples of sequencing notes for construction activities impacting waters of the state.

Stream Bank Construction Sequencing Example

Such as riprap placement, other revetment, placement or repair of structures, etc. (Figure 3-A)

1. Provide minimal clearing to stream for equipment access;
2. Place EPSC measures;
3. Place instream diversion. Instream diversions can be constructed of sandbags, jersey barrier, conventional riprap, sheet piling, or other materials (Figure 3-A);
4. Pump out water inside of diversion, continue to pump the water out as needed;
5. Repair bank or repair intake structure or bridge pier;
6. Remove instream diversion;
7. Apply permanent stabilization measures to disturbed areas; and
8. Remove EPSC measures once disturbed soils are stabilized.

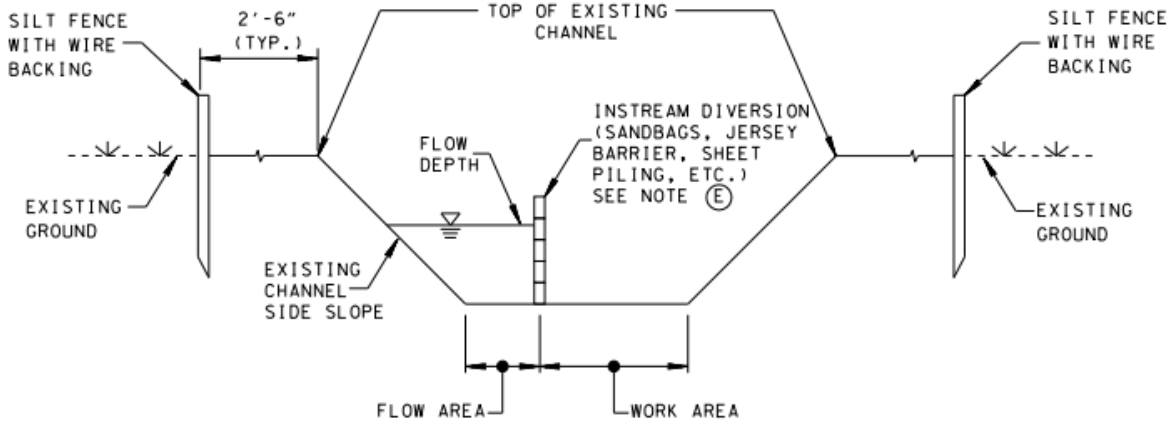


Figure 3-A: Instream diversion details. Source: TDOT - EC-STR-30.

Culvert Crossing Replacement Sequencing Example

Using a pump around diversion may have similar sequencing for a diversion channel or pipe (Figure 3-B)

1. Place EPSC measures adjacent to stream;
2. Place sandbag berm in stream channel before and after work area;
3. Install pump around diversion, pump and pipe from the upstream side to the downstream;
4. Remove any water from the construction area by pumping into a sediment filter bag located downgradient of the work area;
5. Excavate and remove existing culvert;
6. Replace culvert with a new pipe and backfill. For work in jurisdictional streams, ensure the new pipe allows for passage of normal or high flows to comply with USACE NWP General Condition 9 (Management of Water Flows) and does not restrict aquatic organism passage;
7. Stabilize the inlet and outlet of new crossing;
8. Remove the pump-around diversion and the sandbag berms;
9. Visually inspect the new culvert crossing to see if the pipe is functioning and that no erosion is occurring at the inlet or outlet;
10. Apply permanent stabilization measures to disturbed areas; and
11. Remove EPSC measures once disturbed soils are stabilized.

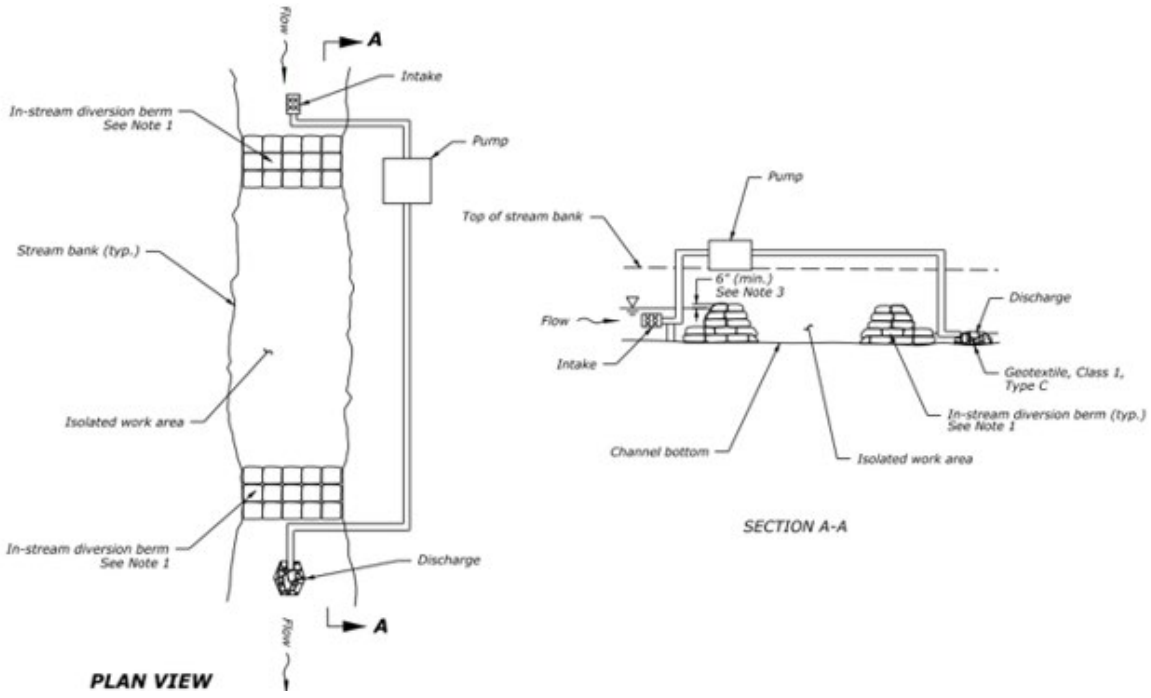


Figure 3-B: Culvert crossing details. Source: FHWA – E628-02 and E628-01for Diversion Berm.

Wetland Impact Sequencing Example

Such as access through a wetland, repair of existing utilities through a wetland, etc. (Figure 3-C).

1. Place timber mats for equipment access to wetland;
2. Place any temporary EPSC measures, if needed;
3. Remove and stockpile the first 12 inches of topsoil from the area to be impacted;
4. Excavate to replace utility line, use a trench box if needed;
5. Replace section of line and backfill;
6. Restore area to pre-construction grade with stockpiled topsoil;
7. Permanent vegetative stabilization using native species of all disturbed areas in or near the wetland must be initiated within 14 days of project completion. Non-native, non-invasive annuals may be used as cover crops until native species can be established;
8. Remove all temporary EPSC measures once disturbed soils are stabilized; and
9. Remove timber mats and vegetate any disturbed soil remaining from construction.

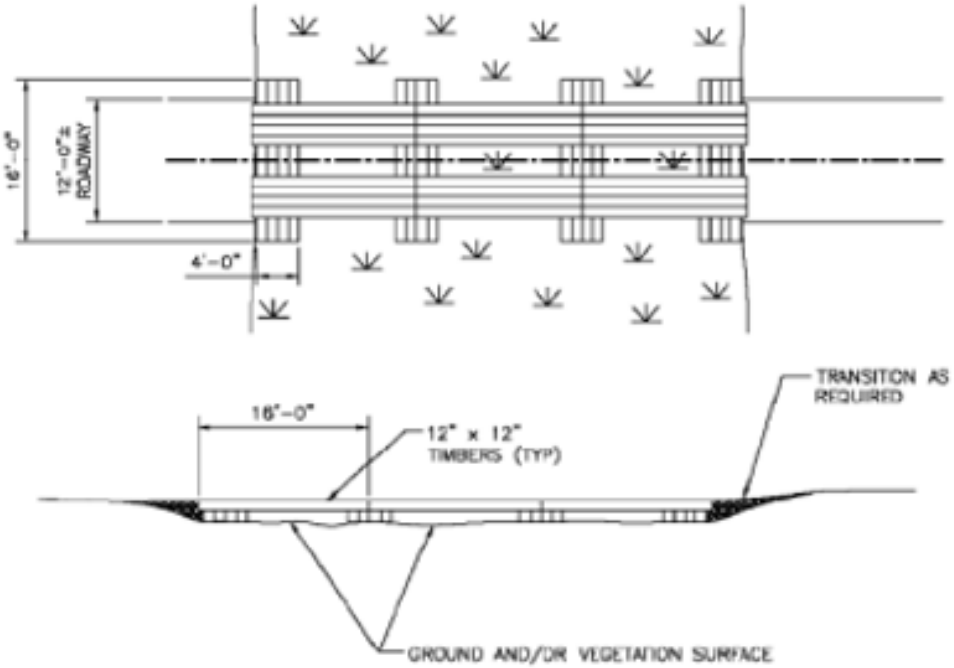


Figure 3-C: Wetland crossing with mats details. Source: USACE (2016).

Lake Impact Sequencing Example

Such as revetment to a dam, building a bridge pier, work in a marina, etc. (Figure 3-D).

1. Install access road;
2. Place EPSC measures, such as floating turbidity curtains;
3. Place a cofferdam;
4. Pump out the water from behind the cofferdam so that it exposes the bottom of the lake. The water can be pumped to a sediment filter bag on land or behind a floating turbidity curtain to minimize sediment from migrating in the lake;
5. Pump as needed to keep area work area in cofferdam dry;
6. Construct the bridge pier;
7. Remove the cofferdam and access; and
8. Remove EPSC measures.

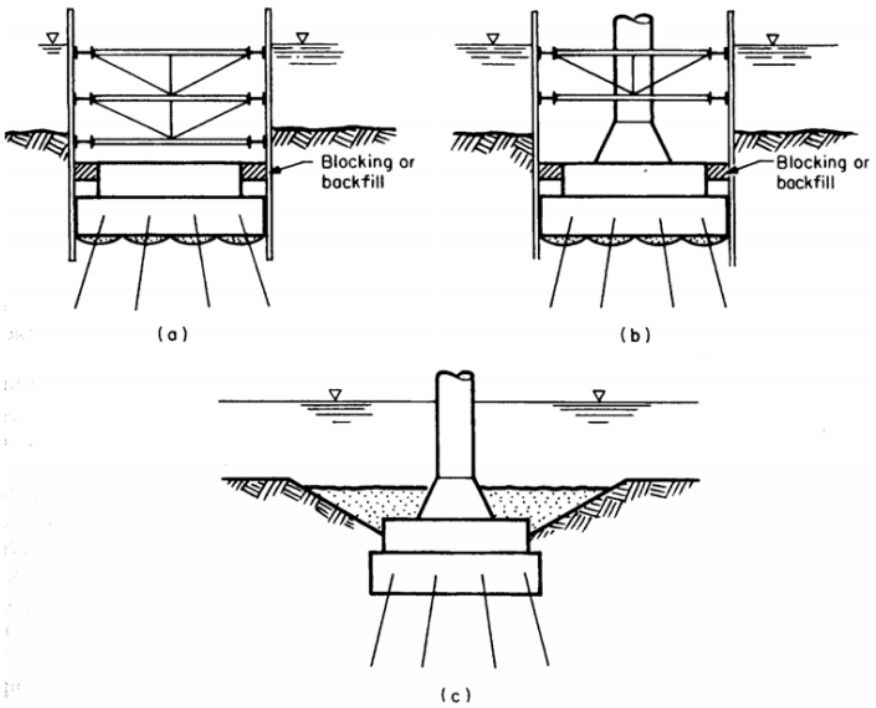


Figure 3-D: Cofferdam construction details. (a). Check blocking, dewater, construct footing block, block between footing and sheet piles. (b) Remove lower bracing, construct pier pedestal, construct pier shaft. (c) Flood cofferdam, pull sheets, remove bracing, backfill. Source: UW Department of Construction Management (2007).

Channel Restoration Sequence and Implementation Example

Conditions: EPSC measures will be placed onsite prior to any groundwork or soil destabilization and remain in place and maintained during land disturbing activities. The new stream channel shall be excavated and stabilized during low-flow conditions. Temporary flow diversion may be used if necessary. Mulch and seeding shall be installed immediately following channel completion in accordance with accepted plans. Trees shall be installed in the first planting season following channel excavation. Water shall be diverted into the new channel **ONLY** after it is completely stabilized, and **ONLY** during a low-water period.

Channel restoration example sequence:

1. Flag edge of the new channel top of bank prior to clearing. Do not clear large trees in position to shade the new channel. Leave as many native trees and shrubs as possible;
2. Excavate the new channel "in the dry" by leaving areas of undisturbed earth (diversion berms) in place at both ends;
3. Shape channel. Any water collected in the new stream channel during construction shall be filtered prior to discharge;
4. Place topsoil, rolled erosion control products, seed, etc. as specified;
5. Remove diversion berms, beginning with the most downstream. Banks and bottom elevation of the old channel shall transition smoothly into the new channel. The elevations of the new channel at both ends of the relocation shall match the elevations of the old channel;
6. Provide a minimum of 72 hours from initial diversion of flow until alteration/filling of the original stream to allow emigration of semi-aquatic organisms; and
7. Install plantings along stream banks as specified in accepted plans.

References:

FHWA. FLH Standard Drawings for the FP-24.
MHFD. (2024a). *Conventional and Soil Riprap Construction Guidance Checklist*.
MHFD. (2024b). *Void-Filled Riprap Construction Guidance Checklist.*
TDEC. *Tennessee Contstruction General Permit*.
TDOT. *Standard Drawings: Erosion Prevention and Sediment Control*.
UM Department of Construction Management. (2007). *CM 420 Temporary Structures: Cofferdams*.
USACE. (2016). *Construction Mat Best Management Practices*.

Chapter 4 - Erosion Prevention and Sediment Control Measures

With site specific EPSC plans, EPSC measures are more effective at reducing, capturing, storing, and treating sediment laden runoff from construction sites. The first and most important step in erosion prevention and sediment control is minimizing soil disturbance and preserving the existing vegetative cover. Well planned projects avoid and minimize adverse impacts, and where disturbance is necessary, the least impactful construction processes are chosen. Final stabilization should be implemented as soon as possible. Broadly, EPSC measures can be classified into three main categories: erosion prevention, runoff velocity control, and sediment control. However, many EPSC measures are multi-functional and reduce sediment pollution by multiple means.

Erosion prevention practices are used to protect exposed soils from erosive forces thereby promoting soil to remain onsite and in place. Runoff velocity controls are implemented to reduce the velocity of runoff for two primary reasons: 1) to limit soil exposure to erosive forces, and 2) so that soil particles have a greater chance of settling before discharged to nearby surface waters. Lastly, sediment controls are designed to reduce or eliminate eroded soils from being transported off the construction site. All forms of EPSC measures will function more effectively when installed and designed with the Five Pillars of construction stormwater management (Section 6.3; Fagan, 2020), innovative methods (Section 6.4), and site specific methods to address common issues (Section 6.5) in mind.

Some EPSC measures are prefabricated and can be purchased from manufacturers for installation on construction sites. Prefabricated devices are manufactured in bulk and are usually devices applicable to areas commonly found on construction sites. Therefore they are not formally designed by the engineer. However, prefabricated devices still require a site specific design aspect. They are to be installed following guidance throughout the manual, per manufacturer specifications, and in accordance with SWPPP details that are specific to the site. That is, site conditions must be appropriate for the prefabricated measure and all site conditions (elevations, stations, etc.) are to be specified in the SWPPP, details that vary from site to site, despite the same product specifications. TDOT's Qualified Product List (QPL) is a resource that includes prefabricated measures and devices approved for use in lieu of a site constructed EPSC measure: (https://www.tn.gov/tdot/materials-and-tests/research---product-evaluation-and-qualified-products-list.html). TDEC does not require EPSC measures to be specifically listed in this handbook or in the QPL; these are resources that contain best practices for maintaining construction site compliance.

4.1 Environmental Impacts

Over the past few decades, copious amounts of research has led to the formulation of design standards and criteria that result in highly effective EPSC measures (in terms of sediment retention onsite) when they are carefully constructed and maintained properly (Moores & Clayton, 2020; Rowlands, 2019). In fact, Moores & Clayton (2020) found that site wide implementation and maintenance of EPSC measures typically resulted in upwards of 90% total sediment retention onsite. However, traditional use of common EPSC measures such as rolled erosion control products (RECPs), silt fences, hydromulches, nettings (typically used on RECPs and wattles), etc., that are highly effective at retaining sediment onsite, can have unintended environmental impacts.

Degradation of plastic EPSC measures (mechanical abrasion, photodegradation, and water-induced erosion of materials) leach chemical additives, microplastics, and macroplastics that are particularly harmful to soil, water, and wildlife (Rich et al., 2020). Byproducts of plastic degradation accumulate in both abiotic and biotic sources (Ali et al., 2021), and because of their persistence in the environment, invertebrates and vertebrates ingest them, leading to accumulation in tissues which then move through the food chain (Horton et al., 2017). Nettings, meshes, and RECPs are usually intended to be left onsite, many of which are not composed of degradable materials. Not only are these products plastic, but they also have great potential for wildlife entanglements, which is perhaps the greatest concern for many agencies across the nation. Further, wildlife entanglement is the most cited environmental harm induced by EPSC measures (Whitman et al., 2025). Other EPSC measures or natural, wildlife-friendly counterparts [materials that use natural fiber or 100 percent biodegradable materials and that use a loose weave with a non-welded, movable jointed netting (USEPA, 2022)] should first be considered. While very limited literature exists regarding the sediment retention capabilities of natural counterparts, agencies across the nation have made visual observations of such measures performing as well or better than their synthetic counterparts (Whitman et al., 2025). Wood and other natural fibers, coconut coir, cotton, and jute are commonly reported natural materials that can replace plastic (Figure 4-A). In addition, while nylon is not a natural material, it may be a better alternative to polypropylene, when measures are to be left in the ground. Nylon is more resistant to degradation than polypropylene and will contribute less microplastics to the environment.



Figure 4-A: Example of a plastic double net coconut rolled erosion control product (left) and a natural, biodegradable counterpart (right) composed of a double jute netting. Source: Theissen and Deem (2025).

Hydromulches are typically dyed green using malachite green, a triphenylmethane, to help operators visualize coverage. However, triphenylmethanes degrade slowly, bioaccumulate, have adverse effects on macroinvertebrates, fish, and various other aquatic and semi-aquatic organisms when used at sufficiently high concentrations (Lyle, 2020). It is therefore recommended that construction personnel purchase hydromulches premixed with dye, as most manufacturers are aware of environmental thresholds and produce hydromulches that are environmentally safe (Theisen and Deem, 2025). Premixed hydromulches are typically considered an environmentally preferred alternative to synthetic RECPs.

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4.2 Erosion Prevention

Erosion prevention practices are ground covers, grading techniques, diversions, and liners that limit erosion through various processes. Ground covers include vegetation, mulch, blankets, and other stabilizers that absorb the energy of raindrop impact, thereby reducing sheet erosion. Grading techniques disrupt continuous slopes, preventing runoff from gaining erosion potential while traversing downslope. Diversions can include slope drains and temporary channels or berms that divert or bypass concentrated flows away from disturbed areas. Grading techniques and diversions help control rill and gully erosion. Lastly, liners such as riprap, RECPs, and vegetation stabilize channels conveying concentrated flows. Refer to Sections 3.3.2, 3.3.2.1, and 3.3.2.2 for definitions and guidance on the different types of riprap: conventional, soil, and void-filled. Stable liners that can withstand shear stress are critical for reducing channel erosion. Implementing these controls helps maintain soil integrity onsite, reduces long-term site maintenance costs, and enhances the overall sustainability of the construction project. It is critical to control erosion by various processes depending on which type of erosion (sheet, rill, gully, or channel) is most likely to be observed due to site specific characteristics. Upon completion of construction, the site must be stabilized. For phased projects, contractors may proceed to the next phase once stabilization measures have been implemented on the previous phase; however, inspections of the stabilized area should continue until vegetative coverage of 70% or greater is achieved, where applicable.

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4.2.1 Diversion Berm or Dike



Source: TNWRRC

Definition and Purpose

Diversion berms are compacted ridges of soil, sometimes paired with a ditch or swale, designed to direct runoff around or through the active areas of construction. They help prevent erosion by routing stormwater around disturbed areas and slopes. They can also promote sediment control by routing sediment laden runoff towards other measures such as sediment basins or traps. These structures are more commonly installed as temporary measures but can remain effective as permanent measures if they are properly stabilized.

Appropriate Applications

Diversion berms are appropriate for construction sites where managing/redirecting stormwater runoff is necessary to prevent erosion, sedimentation, and ponded water. They can be placed above disturbed slopes to redirect runoff away from vulnerable areas, across unprotected slopes to break up flow and reduce erosion potential, or at the base of slopes to capture sediment laden water and direct it to a sediment control measure (ARDOT, 2016; NCDEQ, 2013). When positioned at the perimeter of a site to protect adjacent properties, diversion berms help keep sediment from leaving the construction area (GSWCC, 2016).

These structures are particularly useful for protecting downslope or downgradient areas and maintaining stable working conditions by diverting stormwater before it can cause saturated conditions, erosion, or hinder vegetation establishment in low areas. They are often a cost-

effective solution, as they utilize onsite materials and can be built with standard grading equipment. When used in conjunction with sediment basins, silt fences, or slope drains, diversion berms can significantly reduce the required volume of sediment control measures (City of Albany, 2024; VDEQ, 2024).

Limitations and Maintenance

Diversion berms require proper stabilization to prevent erosion and sediment loss. Without adequate vegetation or structural reinforcement, they can become a source of sediment rather than an EPSC measure. Diversion berms are not suitable in streams. Regular maintenance is necessary, including sediment removal to prevent overtopping and to ensure the berm has the capacity to convey peak flows and velocities.

Planning and Design Considerations

Diversion berms should be sized based on the drainage area that it will receive, the recommended berm dimensions are a minimum top width of two feet and maximum side slope steepness of 2H:1V that are machine compacted and stable (City of Albany, 2024; TDOT). Prior to the start of construction activities, stabilize the berm with seeding and mulch, RECPs, or conventional riprap (ALSWCC, 2018; TDOT). The height of the berm should be able to hold the design storm of 2-year, 24-hour or 5-year, 24-hour with an additional freeboard of 0.3-0.5 feet to prevent overtopping. Account for settling in the berm; this is typically estimated to be 10% (Figure 4.2.1-A). When a berm is installed in conjunction with a slope drain, the top of the berm should be at least six inches above the top of the pipe. Lastly, any trees, brush, stumps or other material which would interfere with the function of the berm should be removed from its alignment and disposed of properly.

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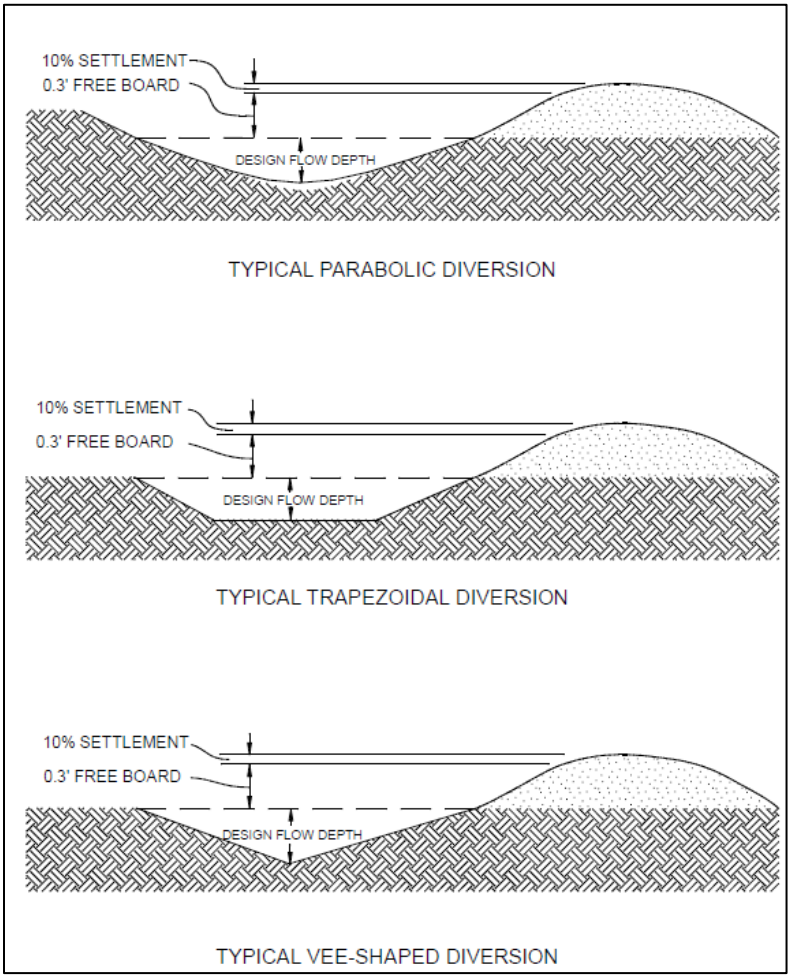


Figure 4.2.1-A: Diversion berm details for various shapes. Adapted from VDEQ (2024).

Diversion berm outlets must be protected and not contribute to erosion. Acceptable outlets include practices such as grassed swales, riprap-lined swales, sediment basins, detention basins, etc. Diversions may also outlet to waterbodies if design flows are not anticipated to cause ecological degradation.

Example Application

Refer to Section 4.3.2.

References

ALSWCC. (2018). *Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

ARDOT. (2016). *Erosion and Sediment Control Design and Construction Manual*.

City of Albany. (2024). *Erosion Prevention and Sediment Control Manual*.

GSWCC. (2016). *Manual for Erosion and Sediment Control in Georgia.*
NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.
TDOT. *Drainage Manual Ch10*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.2.2 Diversion Channel



Source: TNWRRC

Definition and Purpose

Diversion channels, also known as diversion swales, are earthen or lined wet weather conveyances designed to manage stormwater runoff on construction sites. Typically constructed along the perimeter of a construction site, they function by intercepting and redirecting runoff to prevent erosion and protect downstream areas and adjacent properties. These channels can be paired with soil diversion berms or dikes (compacted soil ridges), reinforced mats, vegetative linings, conventional riprap, etc. to control water flow and reduce sediment transport (VDEQ, 2024).

Appropriate Applications

Diversion channels are commonly placed at the top of slopes to prevent runoff from entering disturbed areas or at the bottom to direct runoff toward sediment control measures. These measures are most effective for drainage basins under five acres but can be modified for larger areas. Diversion channels also help protect lower-lying areas from erosion, safeguard vegetation, and control runoff near sensitive areas like sinkholes in karst landscapes. They also help maintain stable working conditions by keeping excessive runoff from entering the

construction site and can be used to divert clean water around environmentally sensitive or disturbed areas.

Limitations and Maintenance

If not stabilized prior to construction activities, runoff will likely erode the channel and become a source of sediment. These channels should be inspected with special attention given to erosion, sediment buildup, and structural integrity. Sediment should be promptly removed from the flow path, and any signs of settling or erosion should be repaired immediately to maintain the original depth and capacity of the channel. By themselves, diversion channels do not have any pollutant removal capability. They must be used in series with an appropriate sediment control measure at the outfall of the diversion channel, while ensuring the outfall is also properly maintained. Reseed or enhance protection on areas that continually fail or fail to establish vegetative cover (VDEQ, 2024). If a diversion dike crosses a roadway, its effectiveness can be compromised, so placement should minimize conflicts with vehicle pathways. Channels should be compacted immediately upon construction and once the construction area is permanently stabilized the ridge and channel should be removed to blend with natural ground cover and appropriately stabilized.

Planning and Design Considerations

Before land disturbance begins, diversion channels should be fully stabilized to prevent erosion and ensure long-term functionality. Construction of diversion swales involves removing all trees, brush, stumps, and obstructions that could interfere with functionality.

The excavated channels often have a parabolic, trapezoidal, or V-shaped cross-section. Cross-sections are to be sized adequately to convey peak flows from the design storm specified in the CGP or local regulations. Refer to Table 2-A for sizing and geometries of square, trapezoidal, V-shaped, and circular cross-sections. Limit side slopes to no steeper than 2H:1V wherever possible. At vehicle crossings, a 3H:1V slope is recommended, with conventional riprap placed with geotextile lining for reinforcement. Refer to Tables 3.3.2.1-A and 3.3.2.1-B and manufacturer specifications for velocity thresholds of various channel linings. The recommended top width of the adjacent berm is at least two feet with a minimum freeboard of 0.3 feet (VDEQ, 2024). To prevent sediment buildup and ensure proper drainage, grades should be uniform or gradually increasing, as sudden changes in grade can lead to sediment accumulation and overtopping.

Runoff from disturbed areas should be directed to sediment traps, basins, or other sediment control measures while flow from undisturbed areas should be discharged in a way that prevents erosion and scour. The discharge location should accommodate the flow from both the diversion and other contributing areas, this is to reduce sediment-laden water from being released off site (NCDEQ, 2013).

Example Application

Refer to Sections 3.3.2.3 and 4.3.2 for an example regarding the sizing of a channel and Section 4.2.6.1 for lining the channel.

References

City of Albany. (2024). *Erosion Prevention and Sediment Control Manual*.
NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.2.3 Diversions: Pipes and Culverts





Source: MPCA (2019)

Definition and Purpose

Diversion culverts and pipes are temporary structures used on construction sites to redirect stormwater away from or through active work areas, keeping the work areas dry and the runoff unpolluted. Supported by measures like sandbags or pumps, they minimize sediment disturbance and direct flow into dewatering or sediment control systems, or cleanly through the active construction area, helping prevent erosion and protecting nearby water bodies (MPCA, 2019)

Appropriate Applications

Diversion pipes are used when sites have large upstream drainage areas. This meaure reduces the volume of stormwater requiring treatment by segregating run-on associated with undisturbed areas from runoff associated with disturbed areas. Diversion pipes are most effective when natural slope conditions allow for gravity-driven flow without the need for pumping.

Limitations and Maintenance

Inspect the pipes for cracks and the outlet for signs of erosion (MCPA, 2019); repair as necessary. During storm events, ensure flow is adequately diverted (i.e., does not bypass the system) and the pipes can adequately convey the flow.

Planning and Design Considerations

The design of diversion pipes must be based on hydrologic and hydraulic analysis to ensure they can safely convey expected flow rates and velocities while minimizing erosion and downstream impacts (TDOT).

The minimum pipe diameter should be selected based on either the 2-year, 24-hour or 5-year, 24-hour event. Pipe diameter is calculated using Eqn 15 in Section 3.3.2.4 and Table 3.3.2.6-A if multiple pipes are necessary. The velocity at which the design storm leaves the pipe, V_p (Eqn 6, Section 2.1.2), is critical in designing an appropriate energy dissipator for the pipe outlet (Section 4.3.4).

When excavation is required, erosion can be controlled using trapezoidal transitions lined with riprap on geotextile fabric, sandbags, or similar materials (TDOT). Riprap should meet size specifications (minimum six inch diameter) and be over-excavated to match ditch grade. Use materials selected to withstand design velocities and shear.

Diversion pipe removal should be conducted carefully to prevent erosion and sedimentation. Inspect the site to confirm that the diversion is no longer needed and that stabilization measures have been installed (Section 3.3.2). Begin by removing the upstream diversion. Then, clear any sediment buildup, extract the pipe, and grade and revegetate all disturbed areas to ensure long-term stability.

Example Application

For sizing diversion pipes refer to Eqn 15, Table 3.3.2.6-A, and the example in Section 3.3.2.4 and 3.3.2.6.

References

MPCA. (2019). *Minnesota Stormwater Manual: Construction stormwater practices – diversion and working in the dry.*
TDOT. *Drainage Manual Ch5*.
TDOT. *Drainage Manual Ch10*.

4.2.4 Grade Controls



Source: USEPA (2021)

Definition and Purpose

Land grading involves reshaping the ground surface to proposed elevations, as approved construction drawings from a professional engineer or other qualified professional indicate. This practice is essential for creating a suitable topography for buildings, roads, parking lots, etc. while also managing stormwater runoff, minimizing soil erosion, and controlling sedimentation during and after construction.

Appropriate Applications

Land grading is applicable to sites where the natural topography must be altered to accommodate new land uses or where shaping the landscape can help reduce erosion risks and lower the cost of sediment control (ALSWCC, 2018). It is particularly useful in uneven terrain or areas with highly erodible soils, as it can help stabilize slopes and manage overland flow (USEPA, 2021). Steep slopes contribute to more erosive forces and sediment loss, while gentle gradual slopes can reduce erosion and allow for easier stabilization.

Limitations and Maintenance

Whenever possible, grading activities should not alter existing overland drainage patterns, as this could lead to elevated runoff velocities (USEPA, 2021). Therefore, pre-existing drainage patterns should be documented before grading activities occur. Areas with high water tables may require subsurface drainage systems to prevent instability (ALSWCC, 2018; KTC, 2015). Grading must not occur where sensitive areas, riparian buffers, and water

resources exist, unless proper authorization has been received. Excavated materials that are stored onsite must be properly stockpiled and stabilized (Section 4.2.6.10) or offsite transport and disposal should be arranged (USEPA, 2021). Offsite disposal areas may be included in permitted areas and therefore may also require proper storage and stabilization.

Construction personnel should inspect all EPSC measures used to stabilize graded areas and divert water around or past graded areas as well as the graded area for signs of gully erosion. Graded areas must be stabilized within 14 days of inactivity or completion of construction or seven days if the graded area is steeper than 35%. This includes onsite and offsite stockpiles (if offsite stockpiles are under permit coverage) that are not actively being used.

Planning and Design Considerations

Effective land grading requires careful planning and design to ensure stability, minimize erosion, and control sediment transport. A qualified professional develops comprehensive grading plans that account for existing soil types, slopes, drainage patterns, and environmentally sensitive areas (USEPA, 2021). The plan typically outlines the grading limits, proposed elevations, and accompanying EPSC measures. Grading limits are not to encroach on property lines, stream banks, vegetation to be preserved, or environmentally sensitive areas without proper authorization. The designer is to specify grades that are not too steep for long-term maintenance (ALSWCC, 2018). Before grading begins, a detailed site survey is often conducted to map existing elevations, drainage patterns, and utilities. To reduce soil disturbance, the grading plan can be phased, limiting the extent of exposed areas at any given time. Furthermore, construction exits, identification of sensitive areas, diversions, and sediment control measures should be in place before grading begins to prevent offsite sediment transport.

Topsoil needs to be stripped, stockpiled in a designated area, and stabilized with vegetation or protective covering for later use in revegetation. Areas designated for clearing are to have all vegetation, roots, and debris removed, while fill areas are to be free of loose soil and oversized rocks to provide a stable foundation. A geotechnical professional can oversee fill placement, ensuring that materials are compacted in layers to achieve the necessary bearing strength for structures. Extended slope lengths and steep slopes should be avoided whenever possible as they can increase rill and gully erosion. On long or steep slopes consider diversions (Sections 4.2.1 and 4.2.2), benches and terraces (Section4.3.1), or other measures at specified intervals (Table 4.2.4-A) to reduce erosion potential (ALSWCC, 2018). For slopes steeper than 3H:1V, additional stabilization methods such as RECPs (Section 4.2.6.6) can be a better method to establish stabilization.

Table 4.2.4-A: Guidelines for spacing breaks in slope. Source: ALSWCC (2018).

Slope (H:V)	Horizontal Spacing (ft)
1:1	20
2:1	40
3:1	60
4:1 and 5:1	80
6:1 to 9:1	120
10:1 or flatter	200

Note: Using professional judgement, horizontal spacing may be adjusted to account for site specific conditions

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites.*

USEPA. (2021). *Stormwater Best Management Practices: Land Grading*.

4.2.5 Slope Drain





Source: TNWRRC

Definition and Purpose

A slope drain is a flexible tubing or conduit designed to safely transport concentrated runoff down a disturbed slope. Slope drains transport runoff collected from a diversion at the top of the slope to an energy dissipator at the toe of the slope, thereby preventing runoff from eroding the face of the slope.

Appropriate Applications

These measures are most impactful when installed along the face of a disturbed slope that is subjected to concentrated runoff; however, they can be installed on practically any slope.

Limitations and Maintenance

To ensure proper function, inspection of slope drains is necessary. It is best to inspect them weekly and after each rainfall to check for blockages, leakage, secure anchoring, and potential undercutting at the inlet and outlet. Furthermore, inspect the diversion at the top of the slope and energy dissipator at the toe of the slope to ensure both are functioning as

designed. Some sources suggest a single pipe should not drain an area larger than five acres (ALSWCC, 2018; ARDOT, 2016; USEPA, 2021) while others specify a maximum of one and a half acres (GSWCC, 2016, TDOT). Ultimately, the designer is responsible for ensuring slope drains can adequately convey runoff without exposing the slope to erosion.


Source: TDEC

Planning and Design Considerations

Slope drains are necessary because there are typically lengthy periods of time between when a cut or fill slope is graded and when final stabilization or a permanent drainage system is installed. During these extended periods of time, disturbed slopes are particularly vulnerable to erosion (VDEQ, 2024).

Slope drain conduit should be made from heavy duty, flexible materials, such as non-perforated corrugated plastic pipe, corrugated metal pipe, or specially designed flexible tubing. Pipes must maintain a uniform diameter along their entire length, in which the diameter is based on its contributing drainage area (Table 4.2.5-A). The drain must be securely anchored with reinforced, hold-down grommets or stakes spaced every ten feet or less to prevent movement under heavy flows. Additionally, the pipe should extend four feet beyond the toe and top of the slope (GSWCC, 2016; TDOT) as further explained in the subsequent paragraphs.

Table 4.2.5-A: Slope drain diameters based on contributing drainage area. Sources: ALSWCC (2018); GSWCC (2016); TDOT; VDEQ (2024).

Maximum Drainage Area (ac)	Pipe Diameter (in)
0.25	8
0.3	10
0.5	12
0.75	15
1.5	18
2.5	21
3.5	24
5	30

At the top of the slope, the slope drain inlet would ideally extend four horizontal feet beyond the crest of the slope and into a diversion berm or similar (Section 4.2.1). The pipe inlet should have a watertight flared end-section or T-section fitting extending into the diversion. The diversion berm often has a minimum height of six inches above the pipe diameter (TDOT; VDEQ, 2024). Other sources suggest a minimum height of 12 to 18 inches above the top of the pipe (ALSWCC 2018; GSWCC, 2016). When the total diversion height is greater than 18 inches at the centerline of the pipe inlet, slope the diversion at a rate of 3H:1V or flatter to connect with the remainder of the diversion. The top of the diversion should be at least two feet in width with side slopes no steeper than 2H:1V (TDOT). Further, the diversion must comply with all specifications in Section 4.2.1. Soil around the inlet must be thoroughly compacted and stabilized using gravel, conventional riprap, or sandbags. These specifications ensure that runoff does not undercut or bypass the slope drain. When the inlet of the slope drain is along a continuous slope, a ditch block (shown in the photo) should be constructed to divert flowing water into the pipe. The ditch block should be of sufficient height to redirect flowing water but not greater in height than the diversion along the top of the slope (TDOT). A stone filter ring or other inlet protection (Section 4.4.6) may be placed at the inlet for additional sediment removal capacity.


Source: TNWRRC

At the bottom of the slope, the slope drain outlet should ideally extend four horizontal feet beyond the toe. This section of horizontal pipe helps reduce the velocity of water and prevent scour. Alternatively, a T-section or fitting/extension may be placed at the outlet to reduce velocity. Often, outlets discharge to level spreaders (Section 4.3.5), conventional riprap apron, or other energy dissipators to further dissipate energy. These practices may be particularly necessary when a four foot horizontal section of pipe cannot be placed due to space limitations at the toe of the slope. Slope drains may also directly discharge into sediment basins, traps, check dams, etc.


Source: USEPA (2021)

Example Application

-Example courtesy of TDOT-

Given:

A proposed ramp has been designed to have a side slope of 2H:1V and a fill height of 20 feet. The ramp is in normal crown with one 16 foot lane, an outside shoulder eight feet wide and an inside shoulder six feet wide. The ramp is 800 feet long at a profile grade of 2%. The designer has determined that the best erosion prevention and sediment control measure for the ramp is a diversion berm with slope drains.

Determine:

The required temporary slope drain quantities for the outside shoulder of the 800 foot ramp.

Solution:

Step 1- Determine the drainage area: The drainage area includes the 16 foot lane and the eight foot shoulder for 800 feet. Thus:

$$\text{Area} = W \times L = (16 + 8) \times 800 = 19200 \text{ ft}^2 = 0.44 \text{ ac}$$

Step 2- Determine the size and number of temporary slope drains: According to Table 4.2.5-A, the maximum drainage area for a 12 inch diameter pipe is 0.5 acres. Since the area to be drained is 0.44 acres, a single 12 inch slope drain is sufficient.

When considering this site, however, it was judged that an 800 foot flow length from the top of the ramp to the drain-pipe would be excessive. Given that the slope along the berm will be 2%, there is a possibility that shear stresses exerted by the flow could become excessive. Thus, based upon engineering judgment, it is decided to use two 8 inch slope drains with drainage areas of 0.22 acres each. This will break up the length of the concentrated flow. Therefore, two 8 inch slope drains, spaced 400 feet apart, are the ideal size and number of pipes.

Step 3- Determine Quantities: The length of each pipe along a 20 foot high 2H:1V slope can be computed:

$$\text{Slope length} = \sqrt{(L^2 + W^2} = \sqrt{(20^2 + 40^2} = 59.7 \text{ ft.}$$

Therefore, the total length of each slope drain is 60 feet, or 120 feet of conduit for both. If T- or L- sections are not used at the outfall, a minimum of four feet of conduit would ideally be added on to the length of each pipe at the inlet and outlet. Thus, the total required length is 136 feet.

References

ALSWCC. (2018). *Erosion Control, Sediment Control, and Stormwater Management on Construction Sites and Urban Areas.*
ARDOT. (2016). *Erosion and Sediment Control Design and Construction Manual*.
GSWCC. (2016). *Manual for Erosion and Sediment Control in Georgia.*
TDOT. *Drainage Manual Ch10*.
USEPA. (2021). *Stormwater Best Management Practices: Temporary Slope Drains*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.2.6.1 Channel and Swale Stabilization



Source: TNWRRC

Definition and Purpose

Channel and swale stabilization can refer to vegetative, bioengineered, conventional riprap, soil riprap, void-filled riprap, paved, or other linings. The purpose of this practice is to establish a non-erosive channel and reduce the velocity of concentrated flow (KTEC, 2015). This section will focus on vegetative stabilize within channels while hard armoring (Section 4.2.6.4) will focus on structural linings.

Appropriate Applications

This practice applies to construction sites that contain concentrated runoff in an open ditch, channel or swale. Typical locations include roadside ditches, channels at property boundaries, channels created by diversion structures, or channels and swales designed as part of a permanent storm water conveyance system for the site. All conveyances of concentrated runoff require stabilization.

Limitations and Maintenance

This measure only includes stabilizing a channel or swale. During inspections, check the channel for debris, scour, or erosion. For vegetated linings, check to see if vegetation is pushed over or lopsided. These are signs the measures do not provide sufficient stabilization. Repair any bare areas and report insufficient stability immediately. For stabilization guidance of such streams, refer to Section 3.3.2.1.

Planning and Design Considerations

Channel cross sections are most commonly trapezoidal, but can also be rectangular, V-shaped, or parabolic. Channels and swales are to be designed to convey the peak flow rate

of the specified design storm in accordance with Section 4.3.2. Because these flows are concentrated, peak velocities and shear stresses may yield erosion within the channel and the outfall. Thus, stabilization is necessary within the channel and an energy dissipator (Section 4.3.4) may be needed at the discharge location.

There are a variety of types of stabilization measures that can be used to line channels and swales:

- Hard armoring (Section 4.2.6.4);
- Rolled erosion control products (Section 4.2.6.6);
- Grasses and vegetation (Sections 4.2.6.11, 4.2.6.12 and Appendix D).

Within each type, there are a variety of choices. For example, within hard armoring, linings may include conventional riprap, gabions, fabric formed revetments, and pavement. As specified in Section 3.3.2.1, Bank Protection, channel and swale stabilization measures must meet permissible velocity and sheer stress specifications. Refer to the previously mentioned sections for construction specifications, permissible velocities, and permissible shear stresses for individual liners. In general, grass lined channels, or vegetated channels are preferred as they more closely resemble natural conditions (KTEC, 2016). In cases where channel velocities, shear stresses, or slopes exceed the capacities of all ideal liners, consider placing check dams (Section 4.3.3) or other measures in the channel to dissipate energy. However, when added measures cannot sufficiently reduce flow velocity and shear within the channel, more extensive linings, such as turf reinforced mats or hard armoring, may be necessary. Impervious linings are generally considered as a last resort.

Ensuring permissible velocities and shear stress are less than peak velocities and maximum shear stresses in the channel is an iterative process (Figure 4.2.6.1-A). First, document the design flow rate, q_p, channel shape, and channel dimensions. Next considering site specific conditions, the surrounding ecosystem, longevity, and budget, determine which lining would be ideal for the given channel. This may involve discussions with an interdisciplinary team, including (but not limited to) engineers, hydrologists, and wildlife biologists. Using Tables in Sections 3.3.2.1, 4.2.6.4, and/or manufacturer specifications, note the Manning's n coefficient, permissible velocity, and permissible shear for the selected liner. Using professional judgment or guidance from the engineer, estimate the depth of flow in the channel during the design flow rate. With this depth, use Manning's equation (Eqn 8) from Section 2.1.2 to estimate an implied peak flow rate, q_i. If q_i is less than five percent different than q_p, the estimated flow depth was acceptable. If not, re-estimate the flow depth with knowledge gained from the preceding steps. Once an acceptable flow depth has been determined, calculate the maximum sheer stress, τ_{max} (Eqns 12-14) from Section 3.3.2.1. If τ_{max} multiplied by a predetermined factor of safety is less than the permissible shear stress

of the selected liner, the linear is appropriate. If the permissible shear stress is greater than τ_{max} multiplied by a predetermined factor of safety, re-select an ideal linear that provides more stability. The process then must be repeated since the Manning's n value will be different.

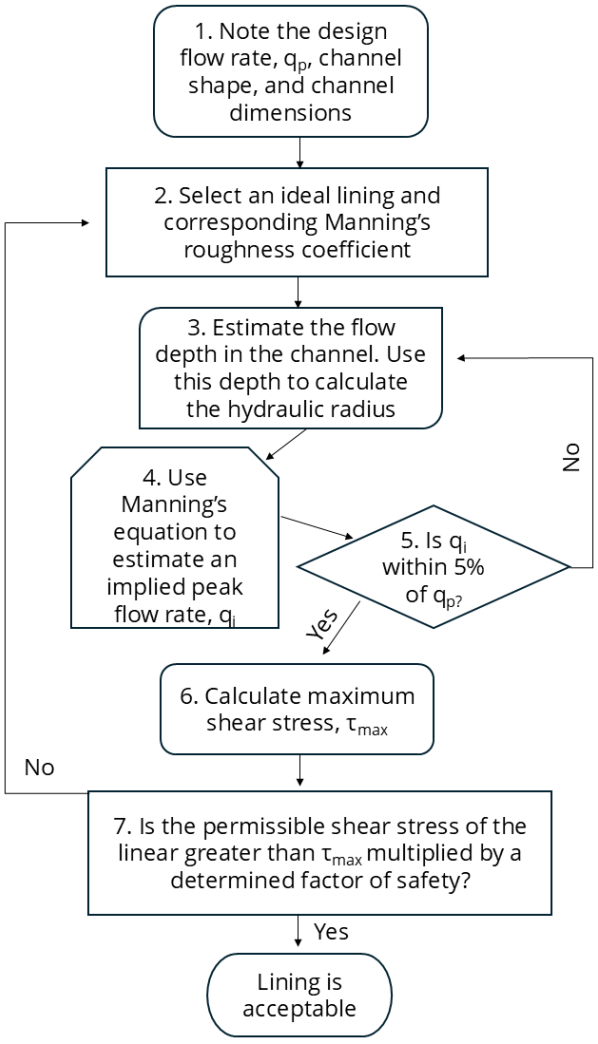


Figure 4.2.6.1-A: Procedure flow chart for determining an acceptable channel or swale lining. Adapted from FWHA (2005).

Remember to follow all construction specifications per individual channel lining. A successfully stabilized channel will not:

- Aggrade or degrade beyond tolerable limits;
- Exhibit signs of erosion;
- Develop sediment bars;
- Form gullies due to the entry of uncontrolled surface flow to the channel;

- Alter the flow capacity of the channel;
- Alter hydrologic properties of the channel;
- Alter the shape of dimensions of the preexisting channel beyond reason; or
- Negatively impact or degrade the surrounding ecosystem beyond reason.

Example Application

Example 1:

Given:

A proposed roadside ditch will extend a distance of 500 feet at a slope of 3.0%. The channel bottom of the ditch is 2 feet and the heigh must be kept below 4 feet. The ditch cross section has 3H:1V side slopes. The engineer has determined that the peak flow rate in the ditch will be 40 cubic feet per second. For aesthetic reasons, a vegetated liner is required for this location.

Determine:

a.) Depth of flow
b.) Velocity of flow
c.) Appropriate channel lining material

Solution:

a.) The depth of flow in the channel may be checked using Manning's equation (Eqn 8, Section 2.1.2) to equal q_p. However, the flow rate will vary based on the roughness, coefficient, so an ideal linear must be selected and tried. Because q_p is moderate, grass likely will not provide sufficient stability. All TRMs provide meet slope requirements, therefore, the least substantial (Type 5a) is tried. Per manufacturer specifications, the Manning's roughness coefficient of a Type 5a TRM is 0.05. Begin with a trial flow depth of 1.5 feet. For a trapezoidal cross section, the flow area, A, wetted perimeter, P, and hydraulic radius, R, are computed (Table 2-A):

$$A = b \times d + z \times d^2 = 2 \times 1.5 + 3 \times 1.5^2 = 9.75 \text{ ft}^2.$$

$$P = b + 2 \times d \times (1 + z^2)^{0.5} = 2 + 2 \times 1.5 \times (1 + 3^2)^{0.5} = 11.5 \text{ ft.}$$

$$R = A / P = 9.75 / 11.5 = 0.85 \text{ ft.}$$

Insert all variables into Manning's equation (Eqn 8, Section 2.1.2) to compute the flow rate corresponding to a depth of 1.5 feet yields:

$$q_i = (1.486 / n) \times A \times R^{0.667} \times S^{0.5}$$

$$q_i = (1.486 / 0.05) \times 9.75 \times 0.85^{0.667} \times 0.03^{0.5}$$

$$q_i = 45.03 \text{ ft}^3/\text{s}.$$

Since the computed flow rate is more than 5% than the design flow rate of 40 ft^3/s, the assumed trial depth is too low. The trial flow depth is varied as shown below:

Trial Depth (ft)	Flow Area (ft^2)	Wetted Perimeter (ft)	Hydraulic Radius (ft)	q_i (ft^3/s)	Difference (%)
1.5	9.75	11.5	0.85	27.83	30.4
1.7	12.07	12.75	0.95	37.05	7.4
1.76	12.81	13.13	0.98	40.135	0.3

A flow depth of 1.76 feet yields an acceptable q_i.

b.) The flow area, A, for a discharge of 40 ft^3/s would be 12.81 ft^2. Using the Continuity equation (Eqn 7), the flow velocity, V, is computed as:

$$V = Q / A = 40 / 12.81 = 3.12 \text{ ft/s}$$

Since the flow velocity is greater than 2 feet per second, grass is not a suitable linear.

c.) Using Eqn 12 (Section 3.3.2.1), the maximum shear stress, τ_{max}, is computed as:

$$\tau_{max} = \gamma \times d \times S = 62.4 \times 1.76 \times 0.03 = 3.29 \text{ lb/ft}^2$$

Since the ditch is on a tangent section of roadway, it will not be necessary to adjust the computed maximum shear stress for curvature (Eqns 13 and 14 from Section 3.3.2.1). According to Table 4.2.6.10-A, a Type 5a linear has a permissible shear stress of 6 pounds per square foot which exceeds the design sheer stress. This linear provides adequate resistance such that a factor of safety is not required.

Example 2:
Given:

A straightened trapezoidal channel will be seeded and has the following characteristics:

$q_p = 20 \text{ ft}^3/\text{s}$
Channel side slopes = 3H:1V
Channel bottom width = 2 ft
Channel height = 3ft
Slope = 0.005 ft/ft

Determine:

d.) Depth of flow
e.) Velocity of flow
f.) Appropriate channel lining material

Solution:

a.) The depth of flow in the channel may be checked using Manning's equation (Eqn 8, Section 2.1.2) to equal q_p. However, the flow rate will vary based on the roughness, coefficient, so an ideal linear must be selected and tried. Because q_p is moderate, grass likely will not provide sufficient stability. Class A1 riprap (n =

0.033) with a trial flow depth, d, of 1 foot is tried. For a trapezoidal cross section, the flow area, A, wetted perimeter, P, and hydraulic radius, r, are computed (Table 2-A):

$$A = b \times d + z \times d^2 = 2 \times 1 + 3 \times 1^2 = 5 \text{ ft}^2.$$

$$P = b + 2 \times d \times (1 + z^2)^{0.5} = 2 + 2 \times 1 \times (1 + 3^2)^{0.5} = 8.32 \text{ ft.}$$

$$R = A / P = 5 / 8.32 = 0.6 \text{ ft.}$$

Insert all variables into Manning's equation (Eqn 8, Section 2.1.2) to compute the flow rate corresponding to a depth of 1 foot yields:

$$q_i = (1.486 / n) \times A \times R^{0.667} \times S^{0.5}$$

$$q_i = (1.486 / 0.033) \times 8 \times 0.56^{0.667} \times 0.005^{0.5}$$

$$q_i = 11.33 \text{ ft}^3/\text{s.}$$

Since the computed flow rate is more 5% less than the design flow rate of 20 ft^3/s, the assumed trial depth is too low. The trial flow depth is varied as shown below:

Trial Depth (ft)	Flow Area (ft^2)	Wetted Perimeter (ft)	Hydraulic Radius (ft)	q_i (ft^3/s)	Difference (%)
1	5	8.32	0.6	1.56	43.3
1.2	6.72	9.66	0.7	16.79	15.6
1.3	7.67	10.22	0.75	20.16	0.8

A flow depth of 1.3 feet yields an acceptable q_i.

b.) The flow area, A, for a discharge of 20ft^3/s would be 7.67 ft^2. Using the Continuity equation (Eqn 7), the flow velocity, V, is computed as:

$$V = Q / A = 20 / 7.67 = 2.61 \text{ ft/s}$$

Since the flow velocity is greater than 2 feet per second, grass is not a suitable linear.

c.) Using Eqn 12 (Section 3.3.2.1), the maximum shear stress, τ_{max}, is computed as:

$$\tau_{max} = \gamma \times d \times S = 62.4 \times 1.3 \times 0.005 = 0.41 \text{ lb/ft}^2$$

Since the ditch is on a tangent section of roadway, it will not be necessary to adjust the computed maximum shear stress for curvature (Eqns 13 and 14 from Section 3.3.2.1). Tables 3.3.2.1-A and 3.3.2.1-C provides maximum allowable flow velocities and shear stresses for riprap; however, it is more ideal to check calculations against manufacturer specifications. Assuming a factor of safety is not needed a maximum velocity of 2.61 feet per second and shear stress of 0.41 are both less than the permissible values for riprap class A1 (permissible velocity of five feet per second and permissible shear stress of 3 pounds per square foot). It appears riprap may be an overdesign for this channel. Therefore, a rolled erosion control product is checked to see if it may be more ideal than riprap (see example in Section 4.2.6.6).

References

FHWA. (2005). *HEC 15: Design of Roadside Channels with Flexible Linings.*

KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites*.

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual.*

TDOT. *Drainage Manual Ch5.*

4.2.6.2 Compost Blanket



Source: Full Circle Mushroom Compost

Definition and Purpose

A compost blanket is a layer of composted organic material applied to disturbed soils on construction sites to prevent erosion, improve soil stability, and support vegetation growth. Acting as a natural mulch, a compost blanket protects the soil by minimizing raindrop impact, retaining moisture, promoting infiltration, and regulating temperature, which enhances plant establishment. Made from materials like yard trimmings, food scraps, and biosolids, it also improves soil health by boosting microbial activity, aeration, and nutrient availability (USEPA, 2012).

Appropriate Applications

Compost blankets can be applied over any exposed or bare soil area. Their ability to be applied directly to the soil surface makes them especially useful for difficult terrain and projects where immediate stabilization is needed, such as those occurring late in the growing season or in areas with nutrient deficient soils. They remain effective over poor soils, rocky terrains, and steep slopes, where other measures become less effective. Compost blankets can be a temporary or permanent measure. When applied as a permanent measure, incorporating seed mixes to the compost is encouraged to promote vegetation growth (VDEQ, 2024). As a temporary measure, cover or nurse crops may be included to the blanket (NCDEQ, 2013).

Limitations and Maintenance

Compost blankets should not be used in areas where nearby waterbodies have high nutrient levels, as compost blankets could contribute more to associated high nutrient impairment (NCDEQ, 2013). Additionally, compost blankets are not suitable in locations with concentrated or channelized flows unless paired with additional erosion control or runoff control measures. Also, they should not be exposed to vehicle traffic (VDEQ, 2024). On steep slopes, additional stabilization methods such as netting or confinement systems may be necessary to prevent displacement (USEPA, 2012). Regular inspections are essential, particularly after heavy rainfall, to identify any damage or areas where vegetation has failed to establish. If sections of the blanket are disturbed, they should be repaired by reapplying compost and seed to restore coverage. In cases where runoff continues to compromise the integrity of the blanket, implementing additional measures may be required to ensure long-term effectiveness.

Planning and Design Considerations

Compost is an organic material created through a controlled biological decomposition process that occurs under aerobic conditions that provides various biological, chemical, and physical benefits for plants and the surrounding soil (USEPA, 2012). Only compost products that meet all federal regulations are permitted for use. Therefore, products certified by the United States Composting Council's Seal of Testing Program are recommended (www.compostingcouncil.org). These products are regularly analyzed using specified test methods, provide product standards, and are easily comparable to one another (NCDEQ, 2013). While Tennessee does not impose state DOT standards, the neighboring states of North Carolina and Virginia do; Table 4.2.6.2-A provides a summary of the required chemical, physical, and biological compost specifications from these states and provide guidance on compost product specifications that are likely to function adequately in a similar climate.

Table 4.2.6.2-A: Physical, chemical, and biological specifications of compost materials for use in North Carolina and Virginia. Sources: NCDEQ (2013), USSC, and VDEQ (2024).

Parameter	Test Method (USCC)	Unit of Measurement	State	Acceptable Range	
				Vegetated	Unvegetated
pH	TMECC 04.11-A	pH units	NC	5.0 - 8.5	NA
			VA	6.0 - 8.5	NA
Soluble Salt Concentration	TMECC 04.10-A	dS/m	NC	< 5	< 5
			VA	< 5	NA
Moisture Content	TMECC 03.09-A	% wet weight basis	NC	30 - 60	30 - 60
			VA	30 - 60	30 - 60
Organic Matter Content	TMECC 05.07-A	% dry weight basis	NC	25 - 65	25 - 100
			VA	25 - 65	25 - 100
Stability	TMECC 05.08-B	mg CO_2-C per g OM per day	NC	< 8	NA
			VA	< 4	< 4
Maturity	TMECC 05.05-A	% seed emergence	NC	100%	90 - 100%
			VA	> 80	NA
Physical Contaminants	Biological Assays	% dry weight basis	NC	< 1	< 1
			VA	< 0.25	< 0.25
Particle Size	TMECC 02.12-B	screen size to pass through	NC	Max – 6 inch 3 inch: 99% 1 inch: 90–100%	
			VA	0.75 inch: 65-100% 0.25 inch: 0–75%	

The following guidelines help ensure proper compost blanket installation (NCDEQ, 2013; VDEQ, 2024):

1. Prepare the soil by removing any large obstructions such as trash, dirt clumps, rocks, roots and anything else that may hinder direct contact of the compost blanket and soil;
2. Apply the blanket to the soil uniformly, following application rates in Table 4.2.6.2-B and as specified in the plans. Rates should be based on site specific characteristics (soils, existing/native vegetation and seedlings) as well as climate. The blanket should be applied over the entirety of the sloped area such that no native soil is visible. Ideally, mix seed thoroughly with the compost blanket or if needed, apply the blanket

at time of seed application. Compost blankets can be applied by hand, bulldozer, skid steer, manure spreader, or pneumatic blower (most cost effective on steep slopes);

3. Install the blanket at least 10 feet extend beyond the top of the slope. Entrench the ending into existing vegetation or soil to ensure runoff does not undercut the blanket; and
4. Blankets installed on slopes greater than or equal to 4H:1V shall be tracked while blankets on slopes 3H:1V shall be stabilized with adequate erosion controlled products. If the slope is 2H:1V or steeper, apply soil stabilization blankets or matting over the compost blanket. Supplement the blanket with filter socks (Section 4.4.9) across the slope to reduce the runoff velocity. Compost berms may be placed at the top of the slope to diffuse concentrated runoff before it reaches the compost blanket.

Table 4.2.6.2-B: Application rates of compost blankets based on local climate. Source: NCDEQ (2013).

Annual Rainfall and Flow Rates	Total Precipitation (in) and Rainfall Erosivity Index	Application Rate (in) for *Vegetated* Compost Blankets	Application Rate (in) for *Unvegetated* Compost Blankets
Low	1-2; 20-90	1-1.5	1-1.5
Average	26-50; 91-200	1-1.5	1.5-2
High	>51, >201	1-2	2-4

Note: Application rates on the lower end of the acceptable ranges should only be used in conjunction with seeding and for compost blankets applied during the prescribed planting season for the local region.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

Full Circle Mushroom Compost. *Compost Blanket*. https://www.fullcirclemushroomcompost.com/projects/mushroom-compost-blanket-pa/.

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.

USCC. Compost – Erosion Control Uses: Compost Blanket Class Specific Guide. Retrieved from: https://www.compostingcouncil.org/.

USEPA. (2012). *Stormwater Best Management Practices: Compost Blankets*.

VDEQ. (2024). *Virginia Stormwater Management Handbook*.

4.2.6.3 Gravel Access Road Stabilization



Source: TDEC

Definition and Purpose

Access or construction road stabilization involves reinforcing temporary access routes, vehicle transportation paths, and parking areas on construction sites to minimize erosion and reduce the need for regrading. By applying materials like gravel immediately after grading, these stabilized routes help maintain safe and durable travel paths for construction traffic, especially during wet weather. This practice ensures that roadbeds remain intact from initial grading through final stabilization, reducing sediment runoff and maintaining site accessibility.

Appropriate Applications

Access road stabilization is applicable wherever temporary or permanent access roads, parking areas, or other travel ways are needed to support construction traffic. This includes stone-base roads and parking areas that may be exposed to erosion and degradation from vehicles, especially during wet weather. It is commonly used on streets, highways, and other traffic areas immediately after grading to maintain stability and minimize soil disruption. Additionally, construction road stabilization helps reduce ground compaction from vehicular traffic and can prevent access and disturbance to sensitive ecological areas (VDEQ, 2024).

Limitations and Maintenance

Inspections should be conducted routinely and after rainfall events to assess surface conditions, check for erosion issues, and ensure drainage structures like roadside ditches and culverts remain clear of sediment and debris. Any areas showing signs of erosion, such as rills or gullies, should be repaired promptly to prevent further degradation. This may require top dressing with stone or gravel (NCDEQ, 2013). Vegetated areas adjacent to stabilized roads should be monitored to maintain healthy plant cover, which helps minimize soil disturbance.

Planning and Design Considerations

Access roads are especially prone to erosion and sediment transport due to constant vehicle movement and exposed soil surfaces. Unstabilized roads often become muddy and unusable during wet weather, causing construction delays and increasing the sites sediment yield (NCDOT, 2013; VDEQ, 2024). Effective road stabilization not only reduces maintenance costs but also ensures safe and efficient construction operations.

To minimize erosion, access roads should be situated following the natural contour of the terrain, avoiding steep slopes, rocky areas, and soils prone to erosion. Aligning roads to follow natural topography reduces disturbance and helps maintain stable drainage patterns (GSWCC, 2016). Where possible, roads should be located on naturally flat or gently sloping areas to reduce grading requirements and minimize the risk of sediment transport. Ridge tops can provide stable road conditions. Additionally, roads built on south- or west- facing slopes are often drier due to sun exposure in intensity, reducing the potential for muddy conditions (VDEQ, 2024).

The roadbed should be cleared of vegetation, roots, and other debris to ensure a solid foundation. Applying a minimum six inch layer of coarse aggregate, such as #57 stone or other coarse aggregate, immediately after grading helps stabilize the surface and prevents erosion. PUG base (Twisted Nail LLC), or other concrete byproducts, may also be used. In areas subject to heavy traffic, increasing the material depth to eight or ten inches may be necessary to prolong roadbed stability (GSWCC, 2016; WVDEP, 2016). Geotextile fabric can also be placed beneath the aggregate to improve stability and reduce the risk of gravel loss, which may be ideal in areas with weaker subgrade conditions.

Road grade plays a significant role in long-term stabilization. It is recommended that road grades do not exceed 10 to 12 percent (NCDOT, 2013), as steeper slopes can lead to gravel loss, rill erosion, and gully formation. Frequent grade changes can help minimize erosion compared to long, continuous slopes. In areas requiring steep grades, switchbacks are preferable to reduce the overall slope length (GSWCC, 2016). Access roads are to be a

minimum of 14 feet in width for one way traffic and 20 feet in width for two way traffic. It is recommended to include a two foot shoulder on each side of the road. Embankment side slopes are most stable when they are 2H:1V or flatter (GSWCC, 2016; NCDOT, 2013). Temporary parking areas are best located on flat or gently sloping terrain with a grade less than four percent (WVDEP, 2016).

Adequate cross-slope grading is crucial to ensure proper drainage to roadside drainage structures, maintain stable road conditions, and minimize erosion. The best suited cross section (Figure 4.2.6.3-A) is often dependent on drainage needs, soil stability, and the expected volume of traffic (VDEQ, 2024):

1. Crowned Fill Section: Use on flat ground where standing water on a road surface may be problematic;
2. Crowned Turnpike Section: Use on low ground roads where fill is not available;
3. Outslope: Use on moderate slopes for low-volume roads and stable soils. Outsloping can be more dangerous in wet and snowy weather;
4. Inslope with Ditch Section: Use on steep hills, areas with fine-textured soils, winter logging roads, and where drainage is necessary; and
5. Crowned and Ditched Section: Use on high-volume roads on steep side hills

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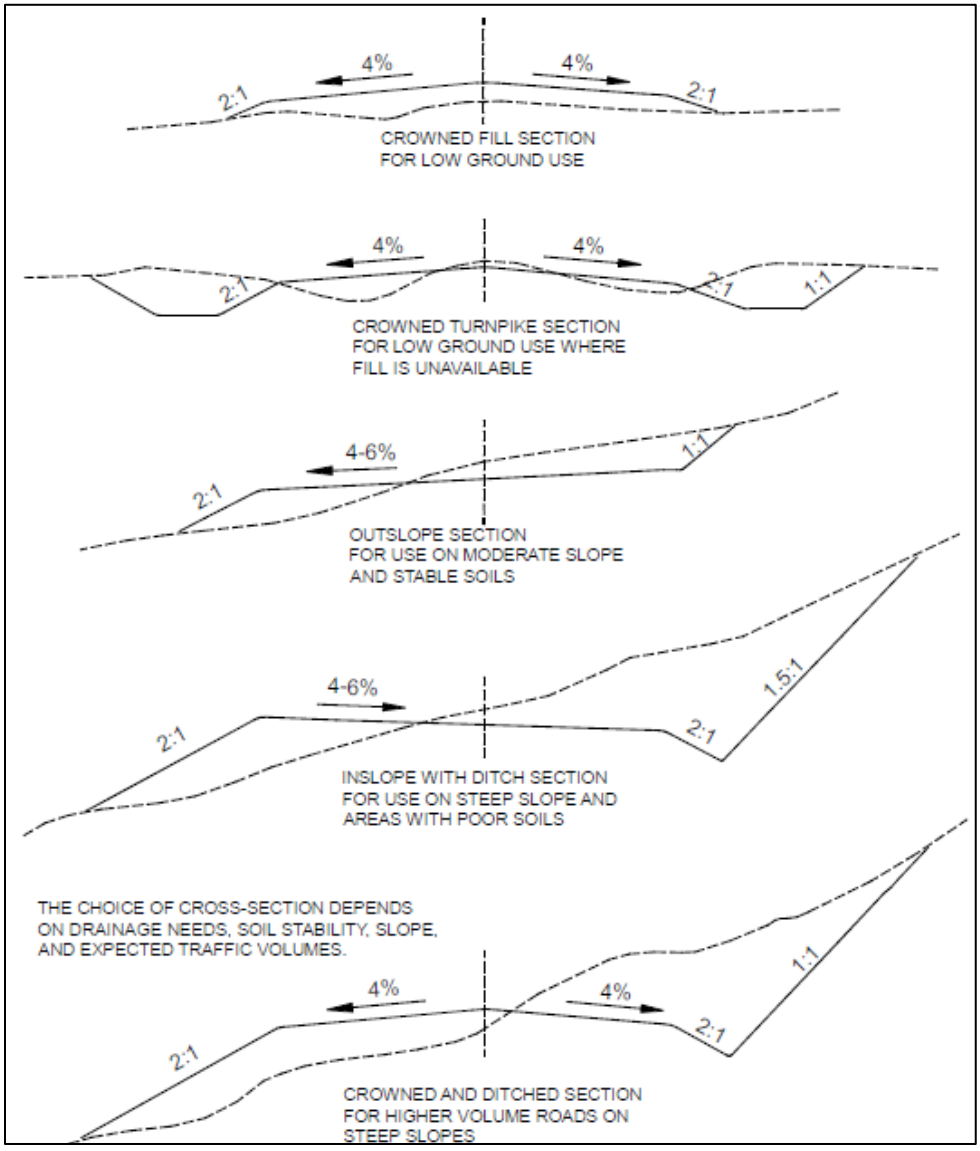


Figure 4.2.6.3-A: Access road cross section details. Adapted from VDEQ (2024).

Roadway cross sections are to convey runoff to roadside ditches, culverts, or other drainage structures. Additionally, roadside ditches, cut and fill slopes, and other disturbed areas adjacent to roadways should be stabilized with vegetation or other erosion control measures as soon as grading is completed.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

AEP. (2018). *FY 2018 General Erosion and Sediment Control and Stormwater Management Specifications for the Construction and Maintenance of Electric Utility Lines.* American Electric Power Service Corporation Environmental Services.

GSWCC. (2016). *Manual for Erosion and Sediment Control in Georgia.*

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual.*

Twisted Nail LLC. *What is PUG Base?* https://twistednail.com/post/what-is-pug-base/#:~:text=Unlike%20regular%20concrete%2C%20PUG%20Base,order%20to%20better%20support%20it.

VDEQ. (2024). *Virginia Stormwater Management Handbook.*

WVDEP (2016). *Erosion and Sediment Control Best Management Practice Manual.*

4.2.6.4 Hard Armoring



Source: TNWRRC

Definition and Purpose

Hard armoring is a structural practice that uses conventional riprap, flagstone, gabions, concrete, or paving blocks to stabilize and convey runoff in channels and swales (Section 4.2.6.1).

Appropriate Applications

This practice applies to construction sites that contain concentrated runoff in an open ditch, channel or swale. Typical locations include roadside ditches, channels at property boundaries, channels created by diversion structures, or channels and swales designed as part of a permanent storm water conveyance system for the site. Hard armoring may be more practical than vegetated or engineered products where (KTC, 2015):

- Channel slopes above five percent predominate;
- Design velocities and shear stresses exceed the capacities of vegetation;
- Prolonged high flows are anticipated;
- Soils are highly erosive and not suitable for vegetation; and
- There is insufficient time to construct, seed, and establish a vegetated lining.

Limitations and Maintenance

This measure only includes stabilizing the channel or swale. During inspections, check the channel for debris, scour, or erosion. Repair any bare areas and report insufficient stability immediately. Hard armoring techniques, while effective at reducing erosion, provide minimal

to no environmental or wildlife benefit as well have reduced aesthetics. Generally, vegetated linings should be considered before hard armoring techniques. When used, long-term maintenance may be needed to remove vegetation growing in areas of hard armor, as this can change the hydraulic capacity and undermine the hard armoring.

Planning and Design Considerations

Channel stabilization is an iterative process using Manning's Equation (Eqn 6, Section 2.1.2) and shear stress (Eqns 12-14, Section 3.3.2.1) to ensure the selected liner can withstand design velocities and shear stresses (Section 4.2.6.1). Hard armoring is most often employed as a channel liner when design flow velocities exceed the tolerance of grass, vegetated, and bioengineered linings. Manning's n values for hard armoring material (Table 4.2.6.4-A) is presented such that the methodology in Section 4.2.6.1 (i.e., Figure 4.2.6.1-A) can be implemented.

Table 4.2.6.4-A) Manning's n values for various hard armoring materials and by flow depth. Source: NCDEQ (2013).

Lining Category	Lining Type	Manning's n values		
		0 - 0.5 ft	0.5 - 2 ft	> 2 ft
Rigid	Concrete	0.015	0.013	0.013
	Grouted Riprap	0.04	0.03	0.028
	Flagstone	0.042	0.032	0.03
	Soil Cement	0.025	0.022	0.02
	Asphalt	0.018	0.016	0.16
Unlined	Bare Soil	0.023	0.02	0.02
	Rock Cut	0.045	0.035	0.025
Gravel Riprap	1-inch D_{50}	0.044	0.033	0.03
	2-inch D_{50}	0.066	0.041	0.034
Conventional Riprap	6-inch D_{50}	0.104	0.069	0.035
	12-inch D_{50}	-	0.078	0.04

A flexible liner, such as conventional riprap is preferred over rigid liners for several reasons (NCDEQ, 2013):

- Conventional riprap liners can be designed to withstand most flow velocities by choosing a suitable gradation;
- Soil and void-filled riprap soil bioengineering techniques (Section 3.3.2.2) can easily be incorporated into conventional riprap design;
- Conventional, soil, and void-filled riprap liners adjusts to unstable foundation conditions without failure;

- Failure of a riprap liner is not as expensive to repair as a rigid liner would be; and
- The roughness of conventional riprap reduces outlet velocity and can reduce flow volume by allowing some infiltration.

Conventional riprap linings are best used on slopes that do not exceed 2H:1V and that are at least one and a half times the thickness of the maximum stone diameter plus the thickness of any required filter or bedding (KTC, 2015; NCDEQ, 2013). Furthermore, velocity and shear stress thresholds for conventional riprap gradations are presented in Tables 3.3.2.1-A and 3.3.2.1-C. Filters or bedding need to be used to prevent piping and should have a thickness of six inches. Additionally, non-woven geotextiles can be used between the riprap and soil surface to prevent soil particle migration. Generally, the non-woven geotextile is recommended to meet the requirements found in AASHTO M288 for a Class 2 separation geotextile (ALSWCC, 2018).

Concrete and flagstone can convey large volumes of water without eroding, are typically not limited by velocity, take up less land area, can be constructed to fit limited site conditions, and require less maintenance (NCDEQ, 2013). However, they are more expensive to design and more labor intensive. Because they have a lower roughness coefficient (Table 4.2.6.4-A), rigid linings yield higher velocities and energies that must be controlled and dissipated at the channel outlet to prevent scour and damage to receiving waterbodies or EPSC measures. Concrete and flagstone (including mortar) linings should be a minimum of four inches in thickness. Channels with rigid liners should not exceed the slopes presented in Table 4.2.6.4-B.


Source: TDEC

Table 4.2.6.4-B: Maximum allowable slopes for channels with rigid liners. Source: NCDEQ (2013).

Lining Type	Height of Lining (ft)	Maximum Permissible Slope (H:V)
Formed Concrete	≤ 1.5	Vertical
Screened Concrete or Flagstone	≤ 2	1:1
Screened Concrete or Flagstone	> 2	2:1
Slip Form Concrete	≤ 3	1:1

Channels may have grassed side slopes and conventional riprap or paved bottoms where velocities are within allowable limits for grass lining along the channel sides, but long-duration flows, seepage, or a high velocity flow would damage vegetation in the channel bottom (NCDEQ, 2013). However, soil riprap or void-filled riprap may be more ideal in this scenario. Paving blocks and gabions have some of the same characteristics as conventional riprap and are often used instead of riprap to fit certain site conditions, such as narrow channels and loose soils.

To ensure properly implemented and stabilized channels, first, the surrounding area should be cleared of trees, stumps, roots, loose rock, and other debris (NCDEQ, 2013). Excavation is then performed to match the grades and dimensions of the channel in the plans as specified by the engineer, and excavated to account for liner and filter thickness (KTC, 2015). For rigid liners, such as concrete, the lining is placed to the required thickness, with attention to proper curing and temperature protection. Transverse contraction joints are used to prevent cracking and are recommended to be spaced approximately 20 feet apart, while expansion joints are placed at a recommended maximum of every 100 feet (NCDEQ, 2013). Conventional riprap liners involve placing either non-woven geotextile or gravel layers immediately after slope preparation. It is then followed by rock placement to form a dense, well-graded mass. Geotextiles and stone bedding are positioned to maintain proper drainage and erosion control, with fabric overlaps secured by anchor pins. Channel outlets may need to be stabilized using energy dissipators, and disturbed areas near and within the ditch should be promptly vegetated or otherwise protected to minimize erosion and reduce water pollution.

Example Application

Refer to Section 4.2.6.1.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites*.

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.

4.2.6.5 Hydroseeding, Soil Binders, and Tackifiers



Source: TNWRRC

Definition and Purpose

Hydroseeding, soil binders, and tackifiers are hydraulically applied soil and slope stabilization measures, sometimes referred to as chemical stabilizers. On occasion, they are applied as powders.

Hydroseeding is a water-based slurry containing mulch, tackifiers, soil amendments and/or seed that can be applied to exposed soils to aid vegetation growth (TDOT). Hydro applications include hydroseeding, hydromulching, bonded fiber matrix (TDOT HECP Type IV - BFMs) and fiber reinforced matrix (TDOT HECP Type V – FRMs) applications. They provide a method of stabilization to slopes that are often difficult to otherwise vegetate.

Soil binders are a large family of products that include plant-based materials, polymeric emulsion blends, and cementitious-based binders that are employed as short-term surface protection material. Polyacrylamides bind soil particles and fiber mulch together thereby increasing the size of particles and making them more difficult to erode (TDOT). Other polymeric emulsion blends, plant-based materials, and cementitious-based materials bind fiber mulch together to form a blanket.

Though tackifiers are an anchoring mechanism, they are often used synonymously with soil binders. Tackifiers are used as a glue that binds straw, mulch, etc. together, making them more difficult to dislodge while protecting soil from wind, runoff, or raindrop impact (MPCA, 2023). Hereafter, soil binders and tackifiers are used interchangeably.

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Appropriate Applications

Hydraulically applied soil stabilization measures are applicable on any site where bare soil is exposed, such as cut and fill slopes and stockpiles. Exposed soils should be stabilized immediately with any method that is most feasible due to budget, season, and vegetation timelines. Hydraulic applications may be more reasonable than mulch, straw, RECPs, etc. on steep slopes, large areas, or where access is limited.

Hydroseeding is a beneficial measure where temporary protection is needed until vegetation can fully establish. Soil binders are effective for short term protection where grading or earth disturbing activities are temporarily halted (CalTrans, 2017).

Limitations and Maintenance

Inspect soil stabilizers for uniform coverage upon hydraulic application and after storm events. Reapply as necessary. Inspect slopes for rill erosion. Rill erosion may occur from underapplied sections at the top of slope where water can infiltrate and traverse the length of the slope. Hydraulic applications should not be used in areas exposed to vehicle traffic or concentrated flow. On steep slopes, HECP Types IV and V are ideal as typical hydroseeding methods are mostly effective on flat slopes with mild precipitation. Soil binders require frequent reapplication (usually at a lesser application rate than the original application), especially after larger storms. Most soil binders (e.g., excluding polyacrylamides) require a minimum curing time before becoming fully effective; therefore, they cannot be applied prior to precipitation. Further, soil binders may not be effective in soils with a high clay content (again, excluding polyacrylamides), highly compacted soils, in low relative humidity conditions, and when temperatures are below 40°F (CalTrans, 2017). Chemical applications may require approval from TDEC and is the responsibility of the engineer to acquire such approvals. Lastly, though hydromulches are often composed of recycled and biodegradable materials (e.g., recycled paper, straw, woodchips), the mulch is typically dyed green using triphenylmethanes. Triphenylmethanes break down very slowly and inadvertently introduce chemicals into the environment that can pose negative ecological impacts when used in sufficiently high concentrations. It is best to purchase hydromulches premixed with dyes as manufacturers are often weary of such environmental impacts and produce their products well below such thresholds. Further, be weary of chemical leachate from soil binders that may cause environmental concerns (e.g., cementitious-based materials).

Planning and Design Considerations

Hydro applications such as hydroseeding and TDOT HECP Types I through V (categories identified in TDOT's QPL 17.020) generally require surface roughening (furrows along contours) before application (CalTrans, 2017). HECP Types IV and V differ from general hydroseeding in that they are applied at a rate of 2,000 - 4,000 pounds per acre and can be

used on slopes up to 1H:1V, whereas hydroseed fiber is not recommended on slopes greater than 4H:1V, is generally applied at a rate of 2,000 pounds per acre, and should be thoroughly mixed with water per manufacturer specifications. Hydro applications should be applied to the surface within 24 hours of seeding and during dry, non-windy conditions. Hydro applications are often dyed green to visually inspect it is being applied to the surface evenly. To ensure the proper application rate, mark off a small section on the ground, and calibrate the sprayer to apply the correct seeding rate for such area, before applying at a large scale. Under relatively normal precipitation, proper coverage with HECP Types IV and V provide soil stabilization and protection of the seed bed for approximately one growing season, while typical hydroseeding applications may provide protection for a few months.

Like hydro applications, soil binders may require soil surface preparation. The soil should be roughened with a crimping or punching type roller. When this is not feasible, the surface can be roughened by tracking (CalTrans, 2017). Depending on the type of soil binder used, soil surface prewetting may be required. Ensure to check prewetting, curing times, and weather restrictions, such that the soil binders are applied with appropriate moisture in the soil. When provided, application rates should conform to manufacturer specifications and should be heaviest at the edges, crests of ridges, and banks to resist wind. Binders should be applied uniformly to the remaining area. Similar to hydro applications, consider using a test plot to calibrate application rates and eliminate the use of dyes if possible. Further, consider any environmental impacts chemical applications may pose to the surrounding ecosystem. When applying liquid soil binders:

- Crown or slope the area to avoid ponding;
- Uniformly and lightly prewet the area or according to manufacturer specifications;
- Apply solution under pressure. Overlap solution six to 12 inches;
- Allow treated area to cure for the time recommended by the manufacturer which is typically 24 hours or more;
- Apply the second treatment before first treatment becomes ineffective, using an application rate of 50% of the original rate; and
- In low humidities, reactivate chemicals by re-wetting as necessary.

Consider Table 4.2.6.5-A when selecting an appropriate type of soil binder for construction sites. Consider where it will be applied: will the binder need a high resistance to leaching or abrasion, and will it be compatible with the existing vegetation? Furthermore, will the binder be used over a long period of time? Consider what soils the binder is being applied on: are the soils compacted, are they high in clay content, or do they possess other properties that may affect the ability of the soil binder to form a surface crust? Lastly, consider soil surface, subsurface, and weather conditions that may indicate how often the binder will need to be reapplied (CalTrans, 2017).

Table 4.2.6.5-A: Properties of soil binders. Amended from CalTrans (2017) with professional guidance.

Parameter	Plant Based Material (Short Lived)	Plant Based Material (Long Lived)	Polymeric Emulsion Blends	Polyacrylamides	Cementitious-Based Binders
Cost	Low	Low	Low	Low	Low
Resistance to Leaching	High	High	Low to Moderate	Low	Moderate
Resistance to Abrasion	Moderate	Low	Moderate	Low	Moderate to High
Longevity	Short to Medium	Medium	Medium to Long	Medium to Long	Medium
Minimum Curing Time (hrs)	9 - 18	19 - 24	0 - 24	Immediate	4 - 8
Compatibility with Existing Vegetation	Good	Poor	Poor	Good	Poor
Mode of Degradation	Bio	Bio	Photo/Chemically	Photo/Chemically	Photo/Chemically
Labor Intensive	No	No	No	No	No
Specialized Application Equipment	Water Truck, Hydraulic Mulcher	Water Truck, Hydraulic Mulcher	Water Truck, Hydraulic Mulcher	Water Truck, Hydraulic Mulcher	Water Truck, Hydraulic Mulcher
Liquid or Powder	Powder	Liquid	Liquid/Powder	Liquid/Powder	Powder
Surface Crusting	Yes[a]	Yes	Yes[a]	No	Yes
Clean-up	Water	Water	Water	Water	Water
Application Rate (lbs/ac)	Variable[b]	Variable[b]	Variable[b]	Variable[b]	4,015 - 12,500

[a]Dissolves upon rewetting and requires (semi-) frequent reapplication (excluding polyacrylamides).

[b]Dependent on product, soil type, and slope.

Short lived plant-based soil binders last less than six months (MPCA, 2023); typically, two to three months according to Tennessee professionals. Common soil binders in this type include guar, psyllium, and starch. Guar gum is from the *Cyammopsis tetragonolobus* plant and is a non-toxic, biodegradable agent. It is to be mixed with water at a rate of 11 to 15 pounds per 1,000 gallons (MCPA, 2023).Psyllium is composed of the finely ground coating of plantago seeds that is applied as either a dry powder or in a wet slurry to the surface of the soil. It dries to form a firm but rewettable membrane that binds soil particles together but permits germination and growth of seed. Psyllium requires 12 to 18 hours drying time. Application rates vary slightly depending on the amount of water in the solution to allow for a uniform slurry flow. Starch is non-ionic, cold water soluble (pre-gelatinized) granular cornstarch. The material is mixed with water and before application. Approximate drying time is nine to 12 hours (CalTrans, 2017; MCPA, 2023). Application rates of soil binders are provided in Table 4.2.6.5-B.

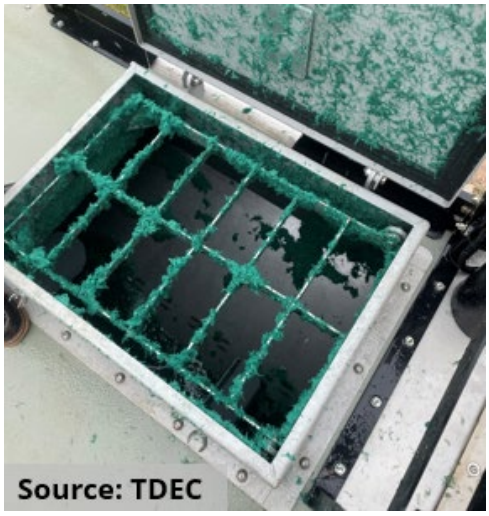
Table 4.2.6.5-B: Application rates for short lived (i.e., less than six months) soil binders. Source: Caltrans (2017) and MCPA (2023).

Soil Binder	Drying Time (hrs)	Application Rate (lbs/ac)
Guar Gum	-	40, 45, 50, 60, or 70 for flat, 4H:1V, 3H:1V, 2H:1V, or 1H:1V slopes
Psyllium	12 - 18	80 - 200
Startch	9 - 12	150

Pitch and rosin emulsion is a plant based soil binder that lasts six to 12 months (MCPA, 2023). Generally, they should have a minimum solids content of 48%. The rosin should be a minimum of 26% of the total solids content. The soil stabilizer should be non-corrosive, water dilutable emulsion that upon application cures to a water insoluble binding and cementing agent. For soil erosion control applications, the emulsion is diluted five parts water to one part emulsion for clayey soils and 10 parts water to one part emulsion for sandy soils (CalTrans, 2017; MCPA, 2023). It should be applied at rates specified by the manufacturer.

Common polymeric emulsion blends include acrylic copolymers and polymers, liquid polymers of methacrylates and acrylates, copolymers of sodium acylates and acrylamides (polyacrylamide), and hydro-colloid polymers. Acrylic copolymers and polymers consist of a liquid or solid polymer or copolymer with an acrylic base, containing at least 55% solids. To prevent foaming during handling and mixing, they may include an anti-foaming agent. The compounds must be readily miscible in water, non-flammable, non-toxic, and should not re-emulsify once cured. Typically, liquid copolymers are diluted at a ratio of 10 parts water to one part polymer and applied at a rate of 1,175 gallons per acre, with air curing taking between 12 to 48 hours. Liquid polymers composed of methacrylates and acrylates act as

tackifiers or sealers, typically containing 40% solids by volume and free from styrene or other potentially harmful compounds. These are diluted according to manufacturer recommendations and applied at around 20 gallons per acre, with a drying time of 12 to 18 hours. Polyacrylamide (PAM) are used for erosion control and are diluted at a rate of 11 pounds per 1,000 gallons of water, with application rates of three to five, five to 10, or 10 to 20 pounds per acre on slopes 5H:1V or flatter, 5H:1V to 3H:1V and 2H:1V or steeper, respectively. Liquid polyacrylamides are used for erosion control and are diluted at a rate of one gallon per 800 gallons of water, with application rates of three, five, and 10 gallons per acre on slopes 5H:1V or flatter, 5H:1V to 3H:1V, and 2H:1V or steeper, respectively. To reduce potential toxicity to aquatic environments, only anionic forms of PAM are permitted: cationic forms have a positive charge that bind to and damage fish gills. Excessive application of PAM can reduce infiltration capacity, have adverse soil binding effects, and may result in a slimy film like deposition (KTC, 2015). Lastly, hydrocolloid polymers, which consist of various dry flowable polymers, are mixed with water and applied at 53 to 62 pounds per acre, drying in about four hours (CalTrans, 2017).


Source: TDEC

Gypsum is a cementitious based soil binder that readily mixes with water and mulch to form a protective crust on the soil surface. Gypsum is ground, calcined and processed to be mixed in a hydraulic seeder and applied at rates 4,000 to 12,000 pounds per acre. Drying time is four to eight hours (CalTrans, 2017). Ensure the use of gypsum allowed per local regulations as it is known to release sulfate (MCPA, 2023).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

CalTrans. (2017). *Construction Site Best Management Practices (BMP) Manual.*

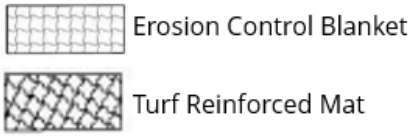
KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites.*

MPCA. (2023). *Minnesota Stormwater Manual: Erosion prevention practices – tackifiers and soil stabilizers.*

TDOT. *Drainage Manual Ch10*.

4.2.6.6 Rolled Erosion Control Products





Source: TDEC

Definition and Purpose

Rolled erosion control products (RECPs) are natural or synthetic woven, sewn, bonded, or manufactured coverings designed to prevent erosion and promote vegetation growth. RECPs include turf reinforcement mats (TRMs) and erosion control blankets (ECBs) that are fabricated into rolls for ease of installation. While both TRMs and ECBs stabilize soils, prevent erosion, and promote vegetation growth they are implemented in different settings due to their differences in material composition and intended longevity.

ECBs are temporary coverings designed to protect bare soil and support the initial establishment of vegetation. They are composed of natural or polymer fibers held together by biodegradable or photodegradable netting. ECBs shield the soil from raindrop impact and wind erosion while promoting moisture retention, reducing soil crusting, and facilitating vegetation growth until plant roots and top growth can sustain long-term soil protection.

Turf reinforcement mats are permanent, non-degradable products engineered to provide long-term erosion protection and structural support for vegetation. TRMs are composed of synthetic, UV-stabilized fibers, filaments, and netting that form a three-dimensional matrix designed to reinforce vegetation by increasing its resistance to hydraulic forces.

Appropriate Applications

RECPs are used to stabilize soil, prevent erosion, and support vegetation establishment in areas where traditional mulching or seeding alone may not be sufficient (USEPA, 2021). Their application depends on site-specific factors such as slope steepness, expected runoff velocity, shear stress, and the need for temporary versus permanent erosion protection.

ECBs are most effective on disturbed areas where vegetation is being established, particularly on slopes 3H:1V or greater and in sheet flow conditions. They are commonly applied on cut or fill slopes, where rapid vegetation growth is necessary to prevent erosion, and over bare soil areas where construction has temporarily ceased (MCPA, 2023a). They help reduce seed loss due to wind, runoff, and wildlife.


Source: TDEC

TRMs are commonly used on excessively long slopes and steep slopes and in channels where flow velocities exceed erosion resistance capabilities of vegetated liners during the design storm (TDOT). TRMs can also be applied along shorelines, at culvert outlets, and in areas prone to scour. Their ability to reinforce vegetation and withstand high shear stresses makes them suitable for areas where natural vegetation alone would be insufficient for long-term stabilization (MCPA, 2023b). Their flexible nature allows them to conform to changing channel shapes due to settlement or erosion, making them a cost-effective and ideal alternative to Hard Armoring (Section 4.2.6.4).


Source: TNWRRC

Limitations and Maintenance

RECPs require thorough site preparation before installation. The subsurface must be smooth and free of debris, large rocks, sticks, and trash, to ensure proper soil contact. Improper staking, shingling, or anchoring, can lead to sagging, lifting, detachment, or displacement of the RECP, thereby reducing its effectiveness (MCPA, 2023a,b). RECPs should be inspected routinely and after large storm events for erosion, washouts, and undermining. Any damage must be repaired by reinstalling or patching affected sections with the same material and

reinforcing the anchoring system. In cases where erosion occurs underneath the RECP, the underlying drainage issue must be corrected before reinstalling the new material.

ECBs, being more temporary, are prone to wind displacement, and in areas with intense storms, they may require additional anchoring or tackifiers to remain in place. Additionally, bulging may occur when seedlings emerge beneath improperly staked or loosely placed ECBs, reducing mat-soil contact. Because of the relatively greater density of TRMs, they can require longer growing periods for the establishment of vegetation as compared to ECBs.

Synthetic RECPs may be a source of environmental microplastics and lead to wildlife entanglements. Use of plastic-free, netless, and natural materials, whenever possible, is recommended.

Planning and Design Considerations

There are five types of RECPs and are categorized based on their functional longevity and physical properties. Types 1 - 4 are ECBs and offer protection ranging from three months to 36 months. Type 5 products are TRMs and are designed for long-term erosion control and provide significantly higher shear stress resistance. Manufacturers may refer to RECPs as TRMs and ECBs or by their type. When used in channels, formal design of RECPs is required since analysis of channels are necessary to determine the anticipated maximum shear stresses along the face of the ditch (TDOT). Use the methodologies presented in Channel and Swale Stabilization (Section 4.2.6.1) to calculate the design velocity and shear stress within the channel and ensure the selected RECP can withstand those forces (Table 4.2.6.6-A). The Manning's n coefficient of RECPs can vary by material and depth of flow, but typically range from 0.02 and 0.04 (NCDEQ, 2013). Defer to manufacturer specifications for Manning's n values when provided.

The effectiveness of RECPs in erosion control (generally on slopes) can be evaluated using RUSLE2 (Eqn 11) as presented in Section 2.2. A key parameter in this assessment is the C-factor, which quantifies the reduction in soil loss due to the erosion control system compared to bare soil conditions (NCDEQ, 2013). The designer may need to conduct soil tests, consult external sources, or use the freely available RUSLE2 model from USDA-ARS to obtain pre-protected C-factors. Because RECPs are made from different materials (Table 4.2.6.6-A), they exhibit varying levels of cost and effectiveness. When examining erosion by itself, the P-factor in Eqn 11 is often considered to be one, indicating no treatment practices. However, through the use of EPSC measures, implementing a P-factor less than one, which reflects the effectiveness of EPSC measures in removing sediment from runoff (NCDEQ, 2013) may be necessary. Like the C-factor, extra effort may be required for accurate P-factor enumeration. Designers are encouraged to use the RULSE2 model from USDA-ARS.

Table 4.2.6.6-A: Classification and specifications for rolled erosion control products. Defer to manufacturer specifications when provided. Source: ECTC (2025).

RECP and Longevity	Type	Product	C-Factor	Material Thickness (in)	Permissible Shear Stress - Vegetated (lb/ft^2)	Permissible Shear Stress - Unvegetated (lb/ft^2)	Maximum Slope Gradient (H:V)
ECB 3 months	1a	Netting or open weave textile	≤ 0.10	≥ 0.03	1	-	5:1
	1b	Netless ECB	≤ 0.10	≥ 0.3	1	-	3:1
	1c	Single-net ECB	≤ 0.15	0.25 - 0.50	1.5	-	3:1
	1d	Double-net ECB	≤ 0.20	0.25 - 0.50	1.75	-	2:1
ECB 12 months	2a	Netting or open weave textile	≤ 0.10	≥ 0.03	1	-	5:1
	2b	Netless ECB	≤ 0.10	≥ 0.3	1	-	3:1
	2c	Single-net ECB	≤ 0.15	0.25 - 0.50	1.5	-	3:1
	2d	Double-net ECB	≤ 0.20	0.25 - 0.50	1.75	-	2:1
ECB 24 months	3a	Open weave textile	≤ 0.10	0.20 - 0.40	2	-	2:1
	3b	ECB	≤ 0.25	0.25 - 0.50	2	-	1.5:1
ECB 36 months	4a	Open weave textile	≤ 0.25	0.20 - 0.40	2.25	-	1:1
	4b	ECB	≤ 0.25	0.20 - 0.50	2.25	-	1:1
TRM Permanent	5a	TRM	-	≥ 0.25	6	2	1:1
	5b	TRM	-	≥ 0.25	8	2	1:1
	5c	TRM	-	≥ 0.25	10	2	0.5:1
	5d	TRM	-	≥ 0.25	12	2	0.5:1
	5e	TRM	-	≥ 0.25	12	2	0.5:1
	5f	High Performance TRM	-	≥ 0.25	14	2	0.5:1

Proper installation of RECPs is essential to ensure their effectiveness in soil stabilization and vegetation growth. The installation process begins with thorough site preparation, which includes grading the surface to create a smooth and loose substrate. All rocks, roots, dirt clods, and debris must be removed to ensure direct contact between the RECP and the soil. If seeding is required before installation, the soil should be prepared by loosening the top two to three inches, incorporating soil amendments like lime and fertilizer (see Section 4.2.6.11), and ensuring even distribution of the seed. In some cases, mulch or a soil-filled matrix may be required before laying the RECP. The surface must be stable and properly compacted to facilitate long-term vegetation establishment (ALSWCC, 2018).

In sloped areas, lay the RECP two to three feet beyond the top of the slope and anchored the ends in an excavated trench at least six inches deep and six inches wide. The material is then pinned at one-foot intervals, backfilled, and compacted to secure the top edge. As the RECP is unrolled down the slope, it must maintain continuous contact with the soil, air pockets or loose areas could lead to failure. Adjacent rolls should overlap by at least two inches, with a secure pinning pattern of staples or stakes placed per manufacturer specifications or every three feet if unspecified. Steeper slopes may require additional anchoring to prevent displacement. Extend the RECP two to three feet beyond the toe of the slope and entrench and anchor the ends at least four inches into the soil and staple at one-foot intervals. Installation details are provided in Figure 4.2.6.6-A.

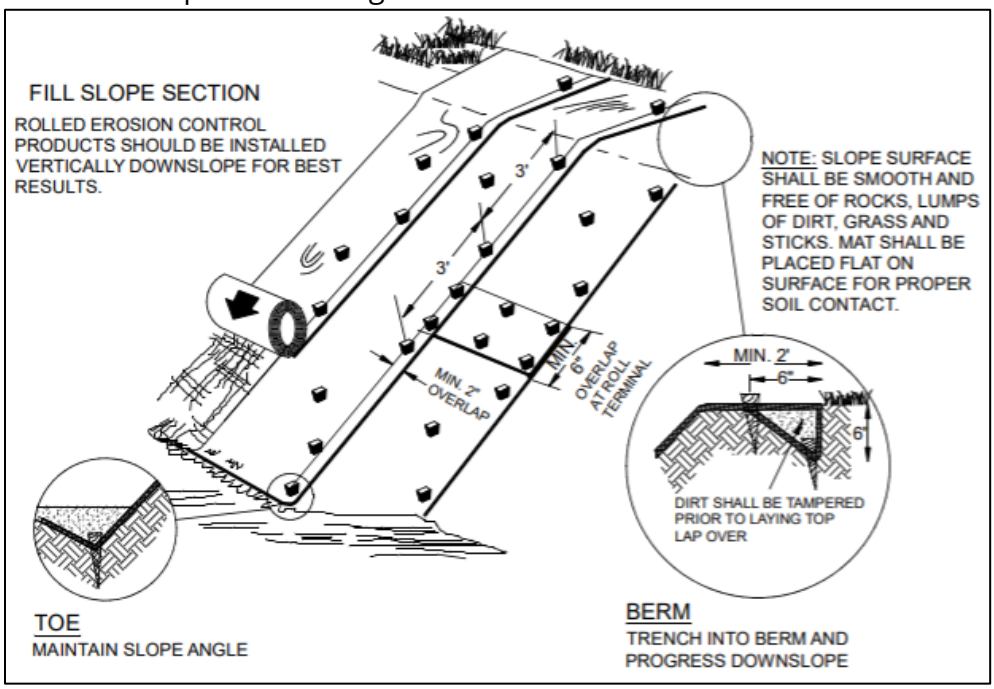


Figure 4.2.6.6-A: Installation details for rolled erosion control products along slopes. Adapted from VDEQ (2024).

In channels RECPs are installed using one of two primary methods, depending on the product specifications and site conditions. In one approach, RECPs are applied directly over a freshly seeded surface, allowing vegetation to establish through mat openings over time. Alternatively, RECPs may be installed first, followed by a topdressing of fine soil and seed to promote immediate rooting within the mat structure. Regardless of method, proper surface preparation is essential, including seeding (if required), grading, and moisture control. RECPs should be unrolled from upslope to downslope, maintaining close contact with the soil and avoiding any stretching or wrinkling. Overlaps should be a minimum of four to six inches in the flow direction and two to four inches laterally. Furthermore, lateral overlaps are to be avoided along the channel bottom; thus, the center of the RECP should align with the centerline of the channel. RECPs must be anchored at least six inches deep at the top and bottom of slopes, and intermittent check slots should be placed at roughly 25 foot intervals along the channel (Figure 4.2.6.6-B). On slopes and in high-velocity flow areas, anchor spacing should be no greater than one and a half to three feet, with longer stakes or staples used in loose or saturated soils. Mat edges and ends should be secured within dedicated anchor slots and backfilled to prevent undermining. Where seeding after installation is specified, a dry, fine-textured topsoil (such as sandy clay loam) should be lightly spread to fill mat voids without covering the entire surface. Final grading and topsoil spreading should be performed using light, rubber-tired equipment to avoid displacement. All disturbed areas must be promptly repaired, and RECPs should be inspected to ensure continuous ground contact, secure anchorage, and adequate coverage, especially at overlaps and terminal points. Defer to manufacturer of ECTC (2025) installation specifications whenever possible.

Metal stakes, wooden stakes, triangular wooden stakes, or U-shaped wire staples can be used to anchor RECPs. It is recommended to use natural materials that will degrade overtime and not need to be picked up once their use is complete. Further, the use of natural materials limits anthropogenic pollutants introduced into the environment. When using metal stakes, use 11 gauge staples that are at minimum six inches in length and one inch in width. Use stakes at least 3/16 inch in diameter with a one and a half inch diameter head. These sizes help ensure effective anchoring (KTC, 2015; NCDEQ, 2013). If maintenance issue persist, consider larger anchoring mechanisms installed more frequently.

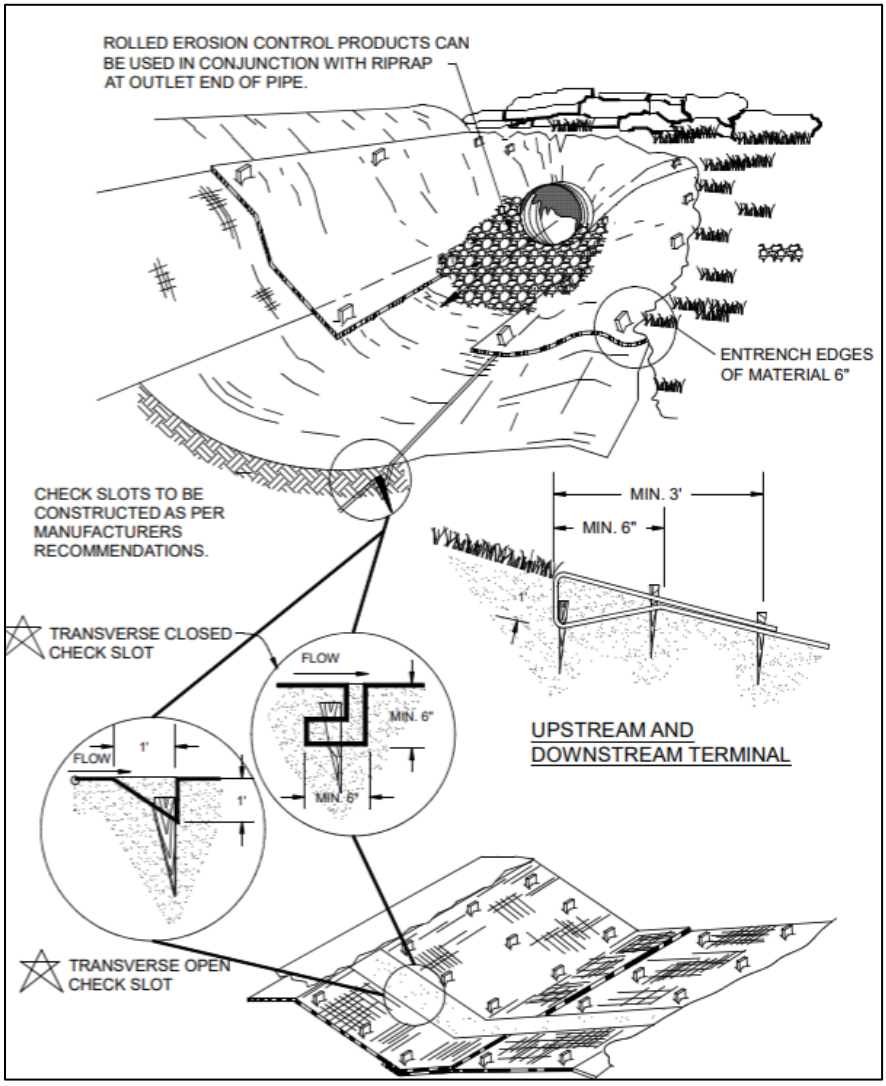


Figure 4.2.6.6-B: Installation details for rolled erosion control products in channels. Adapted from VDEQ (2024).

As mentioned earlier in the manual, there is a growing ambition for using plastic-free materials and environmentally-safe EPSC measures on construction sites (USEPA, 2022). This is particularly true for RECPs, which have the greatest potential to cause wildlife entanglement among EPSC measures, and a high potential for introducing microplastics into the environment (Whitman et al., 2025). Various DOTs (Colorado, Georgia, Minnesota, North Carolina, Vermont, and Washington State) have adopted plastic-free and wildlife-friendly (typically netless) RECPs made from materials such as wood fibers, coconut coir, cotton based products, natural fibers, and jute that have been visually observed to perform as well as or better than their synthetic counterparts in terms of erosion prevention. TRM matting

composed of nylon is a good alternative to polypropylene as it is more resistant to degradation, thereby limiting microplastic (Theisen & Deem, 2025).

Example Application

For channel lining, refer to the example(s) in Section 4.2.6.1.

-Example courtesy of TDOT-

Given:

A 450-foot section of a roadway relocation project in Knox County will require a large cut in a hill adjacent to the site. The soils are silty clay loam. As currently proposed, the slope will be 85 feet high and at a 2H:1V grade. Due to the length and grade of this slope and the condition that the construction site drains to an Exceptional Tennessee Water, there is concern that it be stabilized against erosion as quickly as possible.

Determine:

An appropriate erosion control blanket for the cut slope.

Solution:

Based on the design criteria, a Type 1d, 2d, 3, or 4 would be a suitable ECB for the cut slope. However, given the sensitive nature of the receiving waterway, it is decided to take the additional step of evaluating the potential for erosion from the cut slopes using RUSLE2 (Eqn 9).

Step 1 – Determine the soil loss from the slope if it were not stabilized: Use USDA-NRCS documentation and tables or the freely available software to identify the necessary parameters in RUSLE2. The parameters for a bare soil cut slope are listed with the corresponding reference table from TDOT:

- The R-factor for Knox Co. is 180 [ft × tonf × in × (ac × hr × yr)$^{-1}$] (Table 10A-1);
- Silt clay loam soils yield a K-factor of 0.32 [ton × ac × hr × (hundreds of ac × ft × tonf × in)$^{-1}$] (Table 10-2);
- The slope length can be calculated by the square root of the sum of the squared slope height (85 ft) and the square slope width. The slope width is 2 × 85 = 170 ft since the slope is a 2H:1V ratio. Thus, the slope length is 190.1 ft. Interpolation of LS-factor tables yield a value of 15.48 (unitless) (Table 10A-2);
- The C-factor for bare soil is 1.0 (unitless) (table 10-3); and
- Finally, a P-factor value of 1.3 (unitless) is selected based on the worst case assumption of a compacted smooth surface scraped with bulldozer up and down slope (Table 10-4).

Given these parameters, the average annual soil loss for a bare slope is computed as:

$$A = R \times K \times LS \times C \times P$$

$$A = 180 \times 0.32 \times 15.48 \times 1 \times 1.3 = 1{,}159 \text{ tons / acre / year}$$

The total surface area of the cut slope is 450 ft × 190.1 ft = 85545 ft^2 = 1.96 acres. Thus, the annual soil loss from the cut slope with bare soil would be 2,276 tons per year.

Step 2 – Determine the effect of covering the slope with an ECB: Use USDA-NRCS documentation and tables or the freely available software to identify the necessary parameters in RUSLE2. A Type 3a ECB is initially selected, yielding a C-factor of 0.10 (Table 4.2.6.10-A). Use manufacturer specifications for product details if provided. Herein, assume the P-factor does not change due to implementation of other EPSC measures. Utilizing the same procedures as *Step 1*, the annual sediment load is 227 tons per year.

Step 3 – Use local regulations, engineering judgement, or further knowledge to assess if the ECB provides adequate protection: Herein, no details are provided regarding the pre-development state of the site. Therefore, a comparison between the sediment yield with an ECB and a pre-development sediment yield cannot be made. The annual sediment load is reduced substantially compared to the cut slope without protection; however, a sediment load of 227 tons per year is likely too great for an ETW. Other ECBs suitable for design slope yield larger C-factors, thus an ECB is not suitable for the constraints on this construction site. However, the designer may consider changing the slope of the cut if space permits which would yield a change in the LS-factor and which ECBs are suitable for use.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

ECTC (Erosion Control Technology Council). (2025). *Installation Guide for Erosion Control Products Including Mulch Control Nettings, Open Weave Textiles, Erosion Control Blankets, and Turf Reinforced Mats*.

KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites*.

MPCA. (2023a). *Minnesota Stormwater Manual: Erosion prevention practices – erosion control blankets and anchoring devices.*

MPCA. (2023b). *Minnesota Stormwater Manual: Erosion prevention practices – turf reinforced mats*.

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.

TDOT. *Drainage Manual Ch10*.
Theisen, M., & Deem, L. (2025). *Guidance on 2022 EPA CGP Wildlife-Friendly Erosion Control Solutions.* Virtual Presentation. May 2025.
USEPA. (2021). *Stormwater Best Management Practices: Geotextiles, Matting, and Netting*.
USEPA. (2022). *2022 Construction General Permit*.
VDEQ. (2024). *Virginia Stormwater Management Handbook*.
Whitman, J. B., Schussler, J. C., Perez, M. A. & MAPLE Consulting LLC. (2025). *Use of Sustainable Materials for Erosion and Sediment Control Practices: A Synthesis of Highway Practice*. National Cooperative Highway Research Program Synthesis 643.

4.2.6.7 Sod



Source: TDOT

Definition and Purpose

Sodding is a permanent measure that protects exposed soils by laying a continuous cover of grass. Laying sod establishes immediate ground cover, thereby providing stabilization and preventing erosion. It is particularly useful in situations where seeding is not feasible or where immediate stabilization is required, such as on slopes, drainage channels, swales, or other areas prone to erosion.

Appropriate Applications

Sod can be laid over nearly all graded or cleared areas practically any time of the year (KTC, 2015), although the best times are Spring and Fall. It is one of the best options where immediate, vegetated cover is required. Example applications include:

- Areas where prompt use and aesthetics are important such as lawns and golf courses (USEPA, 2021);
- Steep slopes where vegetative cover may be difficult to establish;
- Grassed swales and waterways; and
- Areas around drop inlets that require stabilization.

Limitations and Maintenance

One of the primary drawbacks of sodding is its higher cost compared to seeding, as well as the challenges associated with obtaining, transporting, and storing sod. To establish a

healthy sod cover, the soil must be adequately prepared, and proper irrigation must be maintained before, during, and after installation. If the sod does not receive enough moisture, especially in the first few weeks, the roots can die back or dry out entirely, leading to sod failure. Regular inspections are necessary during the initial establishment period to monitor moisture levels and grass health, with irrigation needed every few days if there is no significant rainfall. Once established, sod requires ongoing maintenance, including fertilization and mowing. Fertilization schedules differ based on grass type, with warm-season grasses fertilized from late spring to early summer and cool-season grasses from late winter to early fall (NCDEQ, 2013; USEPA, 2021). In cases where sod does not establish properly, consider alternate fertilizer applications or completely remove and replace the sod, with the underlying causes addressed to prevent recurrence.

Planning and Design Considerations

Quality turf can be established with seed or sod. Therefore, it is essential to evaluate whether the benefits of sod justify the expense, especially in situations where rapid stabilization or reduced failure risk is crucial. If sod is used for swales or around inlets, ensure the sod is adequately pinned or staked to the ground and that it can withstand the forces it is subjected to in accordance with Sections 4.2.6.1 and 4.4.4, respectively. Additionally, consider any environmental concerns with sod. Typically, sod has a plastic netting to hold it together and staking materials are either plastic, metal, or wood. Plastic and metal materials will not easily degrade and may remain in the ground for many decades. Plastic-free sod alternatives are available; it is recommended to check with local suppliers for such products.

The type of sod selected should be composed of plants adapted to both the site and the intended purpose (ALSWCC, 2018; NCDEQ, 2013). In general, warm season grasses such as bermudagrass sod would be best in west TN and cool season grasses such as fescue sod would be best in east TN. Both can be used in middle TN, with warmer season grasses in southern middle TN and cooler season grasses in northern middle TN. Bermudagrass is a dormant winter plant and is expected to turn brown for the season, while fescue grass remains relatively green year long. Regardless of type, only high-quality sod of known genetic origin, free from noxious weeds, diseases, or insect infestations should be used (USEPA, 2021). Additionally, sod should be machine cut, containing minimum three-fourths of an inch of soil (not including shoots or thatch), should not be cut in excessively dry or wet weather, and should be cut to size, not sawed (NCDEQ, 2013).

To ensure successful sodding, prepare the site thoroughly before the sod is delivered. Clear surface of trash, woody debris, stones and clumps larger than one inch. Bring the soil surface to final grade and fill low spots in order to avoid standing water (ALSWCC, 2018). The soil should be adequately prepared to address compaction, pH, and fertility issues, as these

factors significantly impact the success of sod establishment. Refer to Section 4.2.6.11 for soil amendment options. For best results, a soil test can be conducted, which can also reduce the need for fertilizers and lime (KTC, 2015).

Use the following installation guidelines to increase sod establishment:

- Sod should be installed within 36 hours of harvest to maintain its viability, as leaving it stacked or rolled can cause heat buildup and damage;
- Do not install sod on gravel, frozen soils, or soils that have been treated recently with sterilant or herbicides. Furthermore, avoid installation during hot months if the site does not have irrigation;
- Moisten and rake the soil surface just before sod is installed;
- Ensure that the sod is in good contact with the prepared soil surface;
- Lay the first row of sod in a straight line with subsequent rows placed parallel to and butting tightly against each other. When smaller rolls are used, stagger strips in a brick-like pattern (Figure 4.2.6.7-A). Ensure the edges of differing pieces of sod abut. Do not stretch or overlap the sod strips;
- Install strips of sod with their longest dimension perpendicular to the slope. On slopes 3:1 or steeper, or wherever erosion may be a problem, secure with pegs or staples;
- After installation, roll sod to provide firm contact between roots and soil, if feasible. This may not be practical on small construction sites or slopes;
- Irrigate the soil after installation; and
- Keep sodded area consistently moist and do not mow until sod is firmly rooted, which is typically two to three weeks.

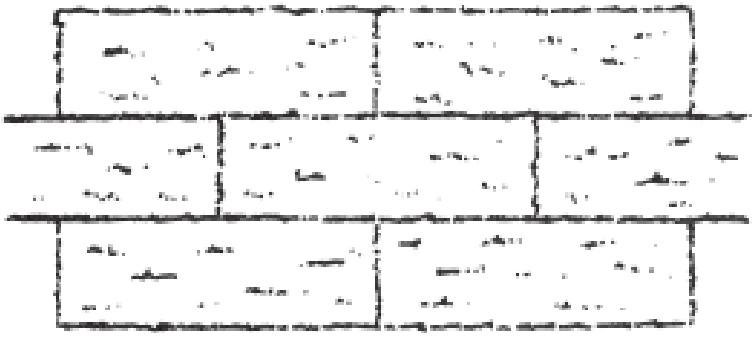


Figure 4.2.6.7-A: Staggered pattern for laying sod. Source: NCDEQ (2013).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites.*

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.

TDOT. *Drainage Manual Ch10*.

USEPA. (2021). *Stormwater Best Management Practices: Sodding*.

4.2.6.8 Stabilization with Plastic Covering: Emergency



Source: TNWRRC

Definition and Purpose

Plastic sheeting is a temporary measure used to protect disturbed soils, slopes, and stockpiles from erosion caused by wind or stormwater. It provides immediate stabilization by creating a physical barrier that prevents soil displacement.

Appropriate Applications

Plastic sheeting is a stabilization measure providing immediate and short-term stabilization by protecting disturbed soils from raindrop impact and wind. This practice may assist in stabilizing stockpiles or slopes that are actively being worked on or when a large storm event is projected or imminent. Furthermore, plastic sheeting is a method to stabilize areas when they cannot be vegetated due to soil conditions, steepness, or seasonal limitations.

Limitations and Maintenance

Use of plastic sheeting on stockpiles and slopes is not intended to prevent erosion in scenarios with concentrated flows or large disturbances. Careful planning and implementation is needed as the impermeable surface can yield high velocity sheet flow causing down gradient erosion issues (City of Albany, 2024). Therefore, proper installation and use of perimeter controls in conjunction with plastic sheeting is recommended. Plastic sheeting is to be inspected regularly for tears, degradation, displacement, anchor failings, vandalism, and undercutting (CalTrans, 2017). Damaged sheeting must be disposed of properly and replaced. Further, plastics degrade and leach chemical additives, microplastics,

and macroplastics, posing environmental concerns and threats. Therefore, plastic covers are not ideal for long term stabilization.

Planning and Design Considerations

Polyethylene, six mil plastic sheeting is advised for use. Once the sheeting is laid over the stockpile or slope, it needs to be weighed down. This can be done with gravel-filled bags, anchor pins, or other weights. Anchoring points are recommended not to exceed six feet in distance. Where multiple sheets are required, it is best to overlap the seams by 12 to 24 inches and secure the seams with tape or a continuous anchoring systems. All edges of the plastic sheeting are to be entrenched a minimum of six inches in the soil. When used as slope stabilization, a diversion berm or dike (Section 4.2.1) at the top of the slope is strongly recommended to divert concentrated flows away from the plastic. This reduces the chance of undermining the plastic on the slope. Additionally, energy dissipation (Section 4.3.4), is advisable around the perimeter of the slope or stockpile toe in order to prevent erosion as the stormwater runs off the plastic (City of Albany, 2024).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

City of Albany. (2024). *Erosion Prevention and Sediment Control Manual*.
CalTrans. (2017). *Construction Site Best Management Practices (BMP) Manual.*

4.2.6.9 Stabilization with Straw or Mulch: Temporary



Source: TNWRRC

Definition and Purpose

Mulching is the process of blanketing exposed soil surface with organic materials like straw, wood chips, corn stalks, wood fiber, or other fibrous residues. The primary purpose of mulching is to protect the soil from raindrop impact and reduce the velocity of overland flow, thereby minimizing erosion and sediment displacement. Mulching also conserves soil moisture, promotes seed germination, acts as a weed barrier, and provides insulation against temperature fluctuations, which helps support healthy vegetation growth.

Appropriate Applications

Mulching is applicable on most construction sites and is a cost-effective solution for temporary stabilization. It is particularly useful when seasonal constraints, such as dry soils, extreme heat, or frost, limit vegetation growth or in areas that cannot be mowed. Further, mulching is an economic option when mulch or wood chips can be made from cleared trees and vegetation. Mulching is most effective when used in combination with seeding or vegetation that, when established, becomes permanent stabilization (USEPA, 2021).

Limitations and Maintenance

During large storms or windy conditions, mulch alone may not provide lasting stabilization. Furthermore, mulching applications are least effective in areas receiving concentrated flow. When these conditions are anticipated, mulch should be applied with an anchoring device, a tackifier (Section 4.2.6.5), in conjunction with other stabilization practices, or hydro applications (Section 4.2.6.5) should be considered.

Mulched areas should be inspected frequently, especially after rainstorms, to check for signs of erosion and dislocation. It is necessary to repair any damaged areas by reapplying mulch

as needed. If the erosion persists, ensure the anchoring device is properly installed or consider adding an additional anchoring mechanism. Maintenance activities should continue until vegetation has established or until construction activities resume.

Planning and Design Considerations

There are a variety of mulch materials available (Table 4.2.6.9-A) and the selection of mulch should be based on site conditions, soil type, vegetation goals, season, and economic factors. Before applying mulch, install sediment control practices, complete the required grading, and, if applying seed, prepare the seed bed. Mulch should be spread uniformly over the disturbed area at rates specified in Table 4.2.6.9-A, covering at least 75% of the disturbed area. When mulch and seed are used in combination, apply the seed before the mulch except in the following scenarios: 1) seed is applied as a part of a hydroseeder slurry containing mulch, and 2) a hydroseeder slurry is applied over straw.

Table 4.2.6.9-A: Mulching materials, application rates, and application notes. Sources: ALSWCC (2018), NCDEQ (2013), and USEPA (2021).

Mulch Material	Application Rate Per (tons/acre)	Application Notes
Straw (no seed)	2 - 3	Spread by hand or machine, requires anchoring.
Straw (with seed)	1 - 2	Spread by hand or machine, requires anchoring.
Wood Chips or Bark	5 - 8	Apply with mulch blower, chip handler, or by hand.
Wood Fiber/Cellulose	0.5 - 1	Use with hydroseeder. Not applicable in hot, dry weather.
Corn Stalks	4 - 6	Apply 4 - 6 inch shredded stalks with mulch blower or by hand.
Pine Straw	1 - 2	Spread by hand or machine, may not require anchoring.

Straw is one of the most commonly used mulching materials on construction sites, particularly in conjunction with seeding, due to its effectiveness in reducing raindrop impact and moderating soil microclimate (KTC, 2015). Straw often comes from wheat or oat and can be applied by hand on smaller sites or blown onto larger areas using specialized equipment. Proper application rates are to ensure soil protection while allowing sunlight and seedlings to penetrate is crucial. It is best to use clean, weed-free straw to minimize the spread of invasive vegetative species. To prevent displacement by wind, straw mulch should be anchored immediately after application (ALSWCC, 2018; VDEQ, 2024).


Source: TDEC

Wood chips or bark are an effective mulching material for areas that will not be closely mowed, such as landscaped areas (ALSWCC, 2018). They decompose slowly and do not require anchoring, making them relatively low maintenance. If soil building or revegetation are part of the plans, nitrogen fertilizer can be applied to the mulch to compensate for the temporary loss of available nitrogen to soil microbes as they break down the carbon-rich mulch (MCPA, 2019). Wood chips or bark can be applied by hand or with a mulch blower.


Source: TNWRRC

Wood fiber mulch, commonly used in hydroseeding applications, is typically made from a combination of wood and paper fibers derived from lumber mill waste, wood chips, or recycled paper products. It is applied as a slurry using a hydraulic applicator and is often mixed with seed and fertilizer. Tacking agents or binders (Section 4.2.6.5) can be added to improve adhesion, especially on steep slopes or critical areas. Wood fiber mulch is most effective for enhancing seed germination and stabilizing soil but is not suitable for erosion control on highly erodible soils or during dry summer and late fall periods.

Corn stalks, pine straw, and hay are less common mulching materials as they may only be available locally or seasonally. Corn stalks are cut or shredded to a size of four to six inches before applied either by hand or blower. Both corn stalks and pine straw are mostly resistant to wind displacement and likely do not need anchoring; however, hay requires immediate anchoring.

Anchoring must be applied to straw (or hay) mulch immediately or to other mulches if inspection notes indicate mulch displacement. Appropriate anchoring mechanisms may include mulch anchoring tools (crimping), nettings (Section 4.2.6.6), or liquid mulch binders and tackifiers (Section 4.2.6.5). Whenever possible, use plastic-free, wildlife-friendly (netless), and biodegradable anchoring mechanisms.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on*

Construction Sites and Urban Areas.
KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites*.
MPCA. (2019). *Minnesota Stormwater Manual: Erosion prevention practices – natural and synthetic mulches.*
NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.
USEPA. (2021). *Stormwater Best Management Practices: Mulching*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.2.6.10 Topsoil Stockpiling



Source: TDEC

Definition and Purpose

Topsoil is the uppermost layers in a soil profile that is rich in organic matter and nutrients vital for plant growth. Typically, topsoil is darker in color and looser in compaction than other soils. Topsoil stockpiling refers to the act of scraping topsoil from a site before mass grading activities and reserving it for later use. It is used to provide a suitable soil medium to support vegetation growth for final stabilization.

Appropriate Applications

Topsoil stockpiling should be utilized on all construction sites where topsoil is available at the surface of the soil. When existing topsoil is not present, importing topsoil may be considered. Stockpiling topsoil is particularly important when vegetation is incorporated as part of final stabilization plans and 1) the existing soil structure is nutrient deficient, overly acidic or basic, or contains toxic substances/pollutants and it is unreasonable or infeasible to amend the existing soil structure (NCDEQ, 2013) and 2) the existing soil depth is too shallow (less than 20 inches depth of solum [upper layer of soil where biologic activity occurs and produce organic material and other minerals]) to provide adequate root depth and soil moisture (VDEQ, 2024).

Limitations and Maintenance

Stockpiled areas require temporary stabilization or covering. Furthermore, sediment control measures such as silt fence (Section 4.4.10) or others must be installed around the downgradient perimeter of the stockpile to prevent sediment migration. Topsoiled areas should be inspected for erosion, depressions or ridges, rocks, and other foreign materials prior to beginning permanent vegetation applications.

Stripping, stockpiling, hauling, and importing topsoil may not be cost effective for all projects. In addition, topsoil may contain weeds, and soil appearance may be misleading as to whether in-situ soils are in fact topsoil. When in question, consult soil specialists to ensure these soils can support vegetation growth.

Planning and Design Considerations

Designers should consider costs, final grade elevations, and topsoil thickness on all projects. Field assessments and soil tests (e.g., ASTM D5268 – "Topsoil Used for Landscaping and Construction Purposes") are recommended to confirm whether there is enough topsoil of suitable quality to justify stripping and stockpiling. Quality topsoil typically has a loamy texture, such as loam, sandy loam, silt loam, sandy clay loam, or clay loam, and should be relatively free of debris, trash, stumps, rocks, roots, and noxious weeds. Ideally, the pH of the topsoil should range from six to seven and a half, and soluble salt concentrations should not exceed 500 parts per million. Liming is an acceptable option to achieve a suitable topsoil pH when the existing pH is less than six but greater than 3.6 (if less than 3.6, the existing topsoil may not be suitable). Organic matter content should not be less than one and a half percent by weight, as it plays a vital role in supporting healthy vegetation. If on-site topsoil is lacking in these qualities, soil amendments and fertilizers, using guidance from Section 4.2.6.11, can be applied to the soil such that it will support vegetation. Otherwise, topsoil that meets the needed characteristics may need to be imported to the site. Another alternative is the use of engineered soil amendments, as determined by ASTM D5268.

It is helpful to evaluate the soil prior to grading to know if the topsoil will be suitable and to know if stripping the soil is a good use of time. Stripping should be conducted in the early phases of construction and should be limited to areas directly impacted by construction activities. Typically, stripping depths range from four to six inches but can vary based on soil type and the specific requirements of the project. It is essential to perform stripping when the soil is neither wet nor frozen to avoid degradation of soil structure (ALSWCC, 2018).

Stockpiling topsoil requires careful consideration of location and stabilization techniques. Stockpiles should be situated away from drainageways, steep slopes, and traffic routes to reduce erosion risks and logistical challenges. The location should ideally be on flat terrain,

and sediment control measures should be installed on the downgradient sides to prevent sediment-laden runoff. Side slopes of the stockpile should not exceed a 2H:1V slope wherever possible. On large sites, smaller, strategically located piles may be preferred for efficient redistribution later.

To minimize soil loss and maintain the quality of stored topsoil, erosion control measures must be implemented based on the duration of stockpile inactivity. For stockpiling storage less than 14 days, sediment barriers such as silt fences or sediment tubes may be sufficient in providing sediment control. For periods longer than 14 days, but activity will resume in the near future, temporary vegetative cover is necessary to stabilize the pile. If the stockpile is to remain dormant for an extended period of time, establishing permanent vegetation is needed to prevent erosion and weed infestation. Refer to Section 4.2.6.11 for reapplication of topsoil to achieve permanent stabilization.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.2.6.11 Vegetation and Landscaping: Permanent



Source: TDEC

Definition and Purpose

Establishing perennial plant cover, such as grasses, shrubs, trees, and legumes, on disturbed or exposed areas to achieve long-term soil stabilization plays a crucial role in reducing the velocity of stormwater runoff, maintaining sheet flow, and promoting filtration and infiltration, which helps prevent sediment loss and soil displacement caused by raindrop impact and surface water movement. By creating a protective cover, permanent vegetation minimizes erosion, enhances site aesthetics, and contributes to improved wildlife habitat.

Appropriate Applications

Permanent vegetation is most applicable where long-term soil stabilization is necessary and where vegetative cover is the most practical or effective solution. It is particularly suited for areas that require durable erosion control. The use of permanent vegetation offers advantages such as lower initial costs and reduced labor compared to other stabilization methods. To ensure effective stabilization, seeding or planting should occur within 14 days of final grading activities or within seven days on slopes exceeding 35%.

Limitations and Maintenance

One of the key advantages of seeding over other planting methods is its cost-effectiveness, flexibility, and lower labor input. However, it comes with challenges such as erosion risk during establishment, seasonal planting limitations, potential weather-related setbacks, and the potential to amend the soil if in-situ soils do not possess desirable qualities. Additionally, some areas may require reseeding if the vegetation fails to establish adequately.

Regular inspections should be conducted to identify areas where vegetation has not taken hold, and corrective measures such as overseeding, reseeding, and re-mulching should be implemented when necessary. Permanent vegetation requires germination and active growth that produces 70% uniform coverage over the disturbed area. When 70% uniform coverage is not obtained, additional seeding, fertilization, or complete replanting following proper seedbed preparation is required, depending on the severity of damaged areas. Soil testing is recommended prior to seeding, as it is a simple test that can aid successful establishment of vegetation. Soil deficiencies, such as improper pH, lack of macro- and micro-nutrients, and low organic matter content, are common issues when vegetation struggles to grow.

Watering is particularly important in dry conditions or during late-season plantings to help vegetation establish a strong root system. In areas with high erosion potential, temporary EPSC measures should be in place until the vegetation is fully established. Fertilization is often required in the second growing season, particularly for turf grasses, and should be applied based on soil test recommendations. However, fertilization may not be required for native species and may increase weed competition. Additionally, vegetated areas should be protected from excessive foot or vehicle traffic, grazing, or other disturbances during the establishment period, with temporary barriers installed if necessary.

A well-defined maintenance plan is essential to ensure the long-term success of permanent vegetation. This plan should address site-specific conditions such as irrigation, mulch replacement, invasive species control, mowing, and corrective actions for vegetation failure.

Planning and Design Considerations

To maximize the probability of successful vegetation establishment, site specific planning is essential. Factors such as soil type, geographic region, and climate, vegetation management requirements, and plant tolerance to site conditions must be evaluated before selecting the appropriate plant species. Refer to the Seed Mix Appendix (Appendix D) for discussion on these topics and example seed mixes.

Site bed preparation is also critically important for vegetation establishment. For introduced species of grass, including turf grasses, the following is recommended for soil preparation:

1. With proper site conditions, stockpile topsoil at the beginning of construction following guidance in Section 4.2.6.10;
2. If feasible, conduct soil tests when the site is ready for vegetation;
3. Using soil test results as well as guidance herein and the Seed Mix Appendix, amend the soil;
4. Rip and/or disk to a minimum depth of 12 inches before applying stockpiled soil. This practice is also advisable on construction sites where no soil material has been stockpiled;
5. Consider engineered soil amendments, biotic soil media, or compost when no topsoil is available;
6. Select a site specific seed mix following guidance from the Seed Mix Appendix;
7. Apply dry mulch at the prescribed rate, hydraulically applied erosion control products, RECPs, or other methods to prevent erosion and seed displacement. Non-plastic erosion control blankets are preferable as they are more environmentally- and wildlife- friendly;
8. Water as necessary if rainfall is not sufficient; and
9. Reapply seed, fertilize, or replant as necessary until 70% uniform coverage of active growth is achieved. Employing no till drill or hydraulic seeding techniques are best practices.

Often soil composition is the reason for difficulty in establishing vegetation at a site. Soil amendments and conditioners can be used to achieve the ideal pH, range of available nutrients, and soil composition. However, topsoiling (Section 4.2.6.10) may be more ideal and cost-effective than applying soil amendments to achieve desirable planting conditions. Care should be taken to minimize the height of topsoil stockpiles to maintain soil health and viability of microorganisms. All candidate topsoil should be tested and in compliance of ASTM D5268 – "Topsoil Used for Landscaping and Construction Purposes". Another alternative is the use of engineered soil amendments, as determined by ASTM D5268.

When topsoil is to be used to aid permanent vegetation, it is essential to properly prepare the subgrade. The subsoil should be disked or scarified to a depth of at least two inches to enhance bonding between the subsoil and topsoil layers. Poor bonding may result in uneven water infiltration and reduced vegetation establishment (KTC, 2015; VDEQ, 2024). Uniform spreading of topsoil to a minimum compacted depth of four inches is recommended for flatter slopes, while two inches may suffice on steeper slopes. Required volumes of topsoil based on spread depth are provided in Table 4.2.6.9-A, which may further assist in cost

comparisons. Topsoil compaction should be light to maintain soil strength and stability without inhibiting root development or increasing runoff.

Table 4.2.6.11-A: Required topsoil volumes per spread depth.

Depth to Spread (inch)	Cubic Yards per 1,000 square feet	Cubic Yards per Acre
1	3.1	134
2	6.2	268
3	9.3	403
4	12.4	537
5	15.5	672
6	18.6	806

Final grading should ensure smooth transitions and eliminate surface irregularities that may impede drainage or contribute to erosion. Following topsoil application, temporary or permanent vegetation should be established promptly to stabilize the soil. Care should be taken to avoid excessive mixing of topsoil and subsoil, as this can compromise soil structure and nutrient availability.

If topsoil is not used, carry out grading according to the approved plan before amending the soil and seeding. Roughen the graded surfaces according to Section 4.3.8.

Soil acidity directly impacts nutrient availability, and most plants thrive in a pH range of six to seven (ALSWCC, 2018; VDEQ, 2024). If soils are too acidic, liming materials such as slow-release agricultural or quick-release micronized limestone or Selma chalk can be applied to raise pH levels. Agricultural limestone should have a calcium carbonate equivalent of at least 90%, and application rates should be determined through soil testing. It is essential to evenly distribute and incorporate lime into at least the top four inches of soil using mechanical means such as disking or chiseling to ensure proper effectiveness (NCDEQ, 2013). Lime typically takes two to three years to fully react with soil, but benefits can appear within a few months. Thus, schedule lime application accordingly. How long those benefits last depends on the type of lime, soil acidity, organic matter, clay content, and how the land is managed (MSU, 2011). However, lime should not be applied to alkaline soils, as excessive alkalinity can hinder plant growth and nutrient absorption. Alkaline soils should be treated with elemental sulfur, gypsum or acidifiers to lower pH.

Where soils lack proper nutrients, ideal N-P-K ratio fertilizers or compost mixes are essential to plant establishment. N-P-K ratios specify the percentage of each nutrient in the fertilizer

where N is nitrogen for green color and growth, P (P_2O_5) is phosphorus for establishment and rooting, and K (K_2O) is potassium to enhance pest and environmental stress tolerance (NCDEQ, 2013). Soil testing provides precise recommendations for nutrient amendments based on plant requirements. If soil tests are impractical, refer to the manufacturer's specifications for application rates. Natural or organic fertilizers derived from composted manure, biosolids, and recycled plant materials may be utilized to improve soil fertility while reducing environmental impact. In sandy soils, which tend to have lower nutrient retention, fertilizer application rates can be increased by 20% to compensate for nutrient leaching (NCDEQ, 2013). This may be more applicable in West Tennessee. Trace elements, such as sulfur, manganese, zinc, boron, chlorine, and molybdenum (and potentially others), are also required for plant prosperity. If these elements are not already included in base fertilizers, they can be obtained individually (NCDEQ, 2013). Similar to pH additives, it is essential to evenly distribute and incorporate fertilizers into at least the top four inches of soil.

Peat, sand, animal manure, rotted sawdust, porous ceramics, biochar, or biosolids can be incorporated into the soil to modify soil texture, structure, and drainage characteristics. Natural amendments, including peat, composted animal manure, and biosolids, increase soil organic matter content and improve nutrient-holding capacity. Fully composted manure and biosolids must meet industry regulations (such as the US Composting Council Seal of Testing Assurance program) before application to prevent environmental contamination. Rotted sawdust and stabilized wood waste can be incorporated, though extra nitrogen may be needed to counteract high carbon-to-nitrogen ratios. In areas with poor drainage or compaction, sand or porous ceramics can be used to improve permeability but must be applied in large volumes to be effective. Proper incorporation of these amendments into at least the top four inches of soil ensures that they provide long-term benefits for vegetation establishment and site stabilization.

After seeding, mulching (Section 4.2.6.9) to protect the seed and the underlying soil is recommended to assist in establishing vegetation. Furthermore, hydroseeding (Section 4.2.6.5), dry seeding, or cultipacker seeding methods may be necessary in conjunction with mulching to retain soil moisture, regulate soil temperature, and minimize erosion during unfavorable weather conditions.

Unlike grasses, the establishment of native seedlings takes several seasons and therefore may require nurse crops in the seed mixes planted and different bed preparation techniques. Notes for the establishment of natives include:

- Select native plant species to match the existing site conditions;

- Ensure all weed competition is controlled before planting the natives. *Proper weed control is the most crucial step in planting native species from seed.* When using herbicides follow all labeled recommendations;
- If using straw mulch on seeded areas straw is to be oats or wheat straw certified to be free from weed seeds. Foreign matter detrimental to plant life and hay, or chopped cornstalks, are not acceptable. Take special care to spread an evenly distributed, thin layer of straw mulch, ensuring no clumps or pieces of bales are not spread;
- If using erosion control blankets, avoid overly dense mats, which may slow or prevent germination;
- Avoid RECPs with plastic netting that can entangle wildlife and lead to the release of microplastics into the environment. Use biodegradable nettings or hydraulically applied erosion control products listed by the US Fish and Wildlife Service as "wildlife friendly" whenever possible; and
- Do not use lime or fertilizer to improve soil conditions if within normal ranges (i.e., no off-the-charts measurements such as found on contamination sites).

In the first growing season, mowing may be needed to prevent undesirable plants from overshading the new seedlings. If undesirable species are present, mow the vegetation no lower than six inches or just above the native seedlings. Spot treat all noxious and invasive plant species within the planting area with appropriate herbicides or hand-rogue individual unwanted plants.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites*.

MSU. (2011). *Facts About Soil Acidity and Lime*.

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.

USEPA. (2021). *Stormwater Best Management Practices: Permanent Seeding*.

VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.2.6.12 Vegetation: Temporary



Source: TNWRRC

Definition and Purpose

Temporary vegetation involves planting fast-growing grasses, legumes, or small grains to provide short-term soil stabilization. By quickly establishing a vegetative cover, temporary seeding helps mitigate dust and erosion potential that arises from exposed soil during construction activities and rainfall. It also aids in preserving soil structure, promoting infiltration, and reducing the velocity of sheet flow. Additionally, temporary vegetation serves as an initial protective layer that prepares the soil for the eventual establishment of permanent plant cover or other long-term erosion control measures.

Appropriate Applications

Temporary vegetation is applicable on construction sites where soil disturbing activities will temporarily cease for 14 or more days. Temporary vegetation may be a common measure on topsoil stockpiles, rough graded areas, temporary sediment basins, temporary earthen structures, bare areas not under active construction, diversion channels, etc.

Limitations and Maintenance

Regular inspections should be conducted to assess seedling emergence and soil stability. Temporary vegetation will be most effective if it uniformly covers 70% of the disturbed area. If bare patches are apparent or if vegetation fails to establish sufficiently, reseeding should be done as soon as possible to prevent erosion. To support germination and early plant growth, it is recommended to loosen soil prior to seeding and apply mulch or straw at the prescribed rate (Section 4.2.6.9) after seeding to help regulate soil moisture and temperature. Adequate watering is also necessary, particularly in hot or dry conditions, but

care must be taken to control watering rates to prevent runoff. Additionally, the area needs to be protected from excessive traffic to avoid soil compaction and disturbance that could hinder plant establishment. Mowing may need to be avoided in order to allow vegetation to develop properly and serve its intended function of erosion control.

Planning and Design Considerations

Temporary seeding is an effective and cost-efficient method for erosion control, particularly in areas not subject to immediate construction activity. Annual plant species, which germinate rapidly and complete their lifecycle within a single growing season, are particularly suitable for this purpose. The successful establishment of temporary vegetation depends on several key factors, including proper site preparation, seed selection, and soil amendments. Before seeding, any necessary erosion control structures should be installed to manage runoff effectively.

The seedbed must be well-prepared, ensuring a loose, uniform soil texture that facilitates seed germination and plant growth. Prepare the seedbed and use soil amendments or topsoil in the same manner as specified in Vegetation and Landscaping: Permanent (Section 4.2.6.11) for temporary vegetation. Avoid seedbed preparation techniques before large rainfall (KTC, 2015).

Seeding success is highly dependent on seasonal conditions and plant species selection. Seeding methods should ensure uniform distribution, with recommended depths varying based on plant species. Follow guidance in the Seed Mix Appendix (Appendix D), for example, plant species and application rates.

In conjunction, or as an alternative method of stabilization, Hydroseeding (Section 4.2.6.5), which combines seed, fertilizer, and mulch in a slurry, can be an effective technique for steep slopes or difficult-to-access areas. Mulching (Section 4.2.6.9) is particularly necessary in harsh conditions, including areas subject to winter seeding, hot or dry climates, those with highly erodible soils, or on steep slopes. Temporary vegetation is expected to provide soil stabilization for up to one year since it survives for only one growing season. After which, permanent seeding or alternative stabilization measures will likely be needed to maintain erosion prevention measures (NCDEQ, 2013; VDEQ, 2024).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites*.

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.

VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.3 Runoff Velocity Control

Controlling the velocity of runoff on construction sites is crucial in preventing soil erosion and minimizing sediment transport. High velocity runoff can dislodge soil particles and carry them into nearby water bodies, contributing to sedimentation impairments. Managing runoff velocity through practices such as check dams, energy dissipators, benching and terracing, etc. helps reduce erosion rates and may enhance infiltration. This reduces the volume of polluted stormwater entering conveyance or drainage infrastructure while slowing the velocity of runoff, a critically important aspect in providing sufficient time for particulates to settle. Furthermore, controlling runoff velocity mitigates the risk of structural damage to construction site infrastructure and neighboring properties caused by uncontrolled water flow. Implementing these measures promotes sustainable construction and perseveration of nearby ecosystems.

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4.3.1 Benching and Terracing



Source: TDEC

Definition and Purpose

Benching and terracing reshape land surfaces based on engineering plans. Benching creates horizontal steps (benches) on steep slopes to stabilize them during construction, while terracing forms level strips (embankments) that interrupt and slow the flow of water, thereby reducing its erosive power. These intermittently sloped surfaces decrease runoff velocity, trap sediment, promote infiltration, and support vegetation growth (Nashville – Davidson County, 2021).

Appropriate Applications

Benching and terracing are best applied on steep (3H:1V or greater) or extended slopes taller than five feet where erosion risks are high and traditional grading is not feasible (TDOT). Such slopes are at greater risk of generating concentrated flow during rainfall. These measures are particularly useful when slopes are composed of cohesive soils, rather than sand or rock (ITD, 2011). For shorter or more gradual slopes, other measures may be more economical.

Benches and terraces require sufficient space for proper widths, maintenance access, and a stable outlet constructed (Hamilton County, 2008). Refer to Table 4.2.4-A for guidelines on spacing for breaks in slopes.

Limitations and Maintenance

Avoid benching and terracing on slopes with sandy or rocky soils, non-cohesive slopes, or slopes lacking structural integrity to support reshaped landforms and construction equipment (ITD, 2011). Decomposing rock and materials with thin, unstable layers can lead to slope failure, especially with excessive water infiltration. Additionally, infiltrated water without proper drainage outlets can saturate the soil, leading to instability and erosion.

Maintenance is critical for success while vegetation is establishing. After significant rainfall, inspect for rills, gullies, and washed-out seed and mulch as they indicate the need for prompt repair. Surface roughening (Section 4.3.8) can enhance terracing but may need to be reapplied after heavy rainfall.

These practices are usually permanent and should be designed by a professional engineer.

Planning and Design Considerations

When planning and designing benching or terracing on slopes, it is important to move beyond aesthetics and focus on functionality. While a smooth, uniform slope might look "finished," it often leads to poor infiltration, increased runoff, and erosion issues. In contrast, roughened, intermittent, or “stair-stepped” slopes are more effective at slowing water, capturing sediment, and establishing vegetation (Hamilton County, 2008).

Incorporating diversion channels or berms (Sections 4.2.1 and 4.2.2) at the top of the slope limits the amount of runoff traversing the slope. Additionally, limiting the overall length of a slope will reduce runoff velocity and erosive power. When slopes are very long, slope drains (Section 4.2.5) or a combination of stabilization practices (Section 4.2.6) may be more beneficial than benches and terraces.

Benching is beneficial for excavating and creating geotechnically stable slopes, either as a method of initial slope construction or final slope grading and stabilization. Design features of benches include (Figure 4.3.1-A):

- Horizontal spacing;
  - Wide enough for a bulldozer (TDOT)
- Vertical cuts;
  - At least 30 inches in soft soil
  - No more than 40 inches in rocky soil (TDOT; VDEQ, 2024)
- Cut ratio;
  - 1.5H:1V or shallower (TDOT)
- Slight backslope on the horizontal cuts; and

  - Allows runoff to drain toward the vertical cut, thereby increasing infiltration (TDOT; VDEQ, 2024)
- Horizontal, longitudinal slopes channeling runoff to a stable outlet.
  - 2 to 3% (MDE, 2011)

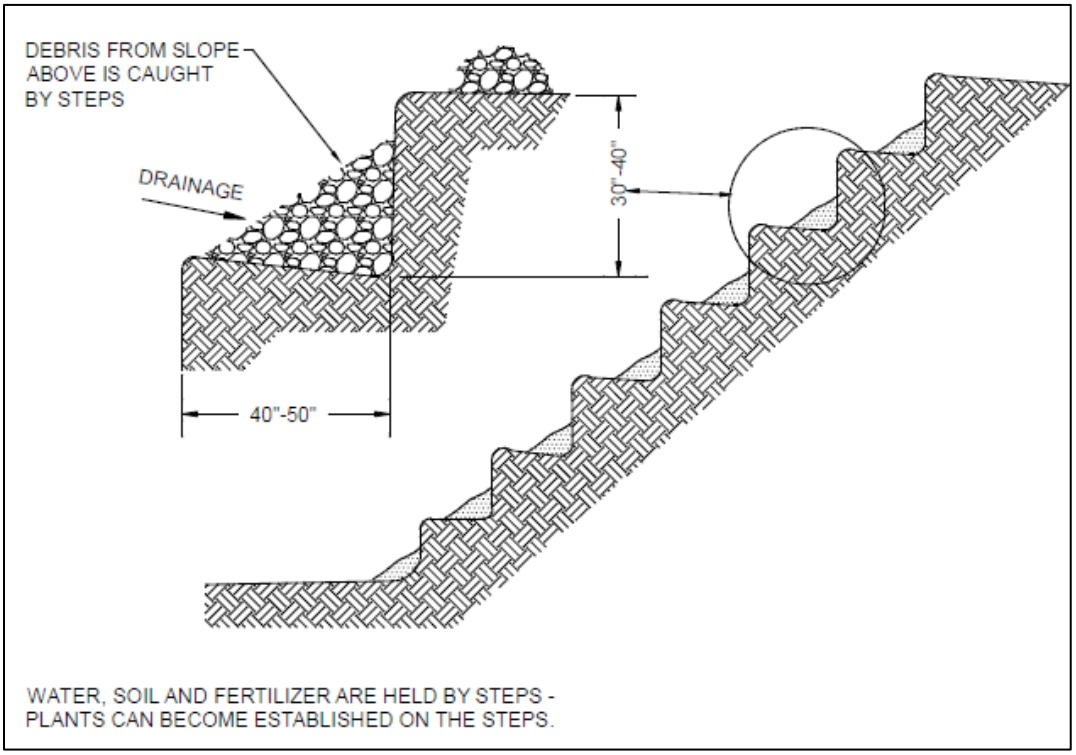
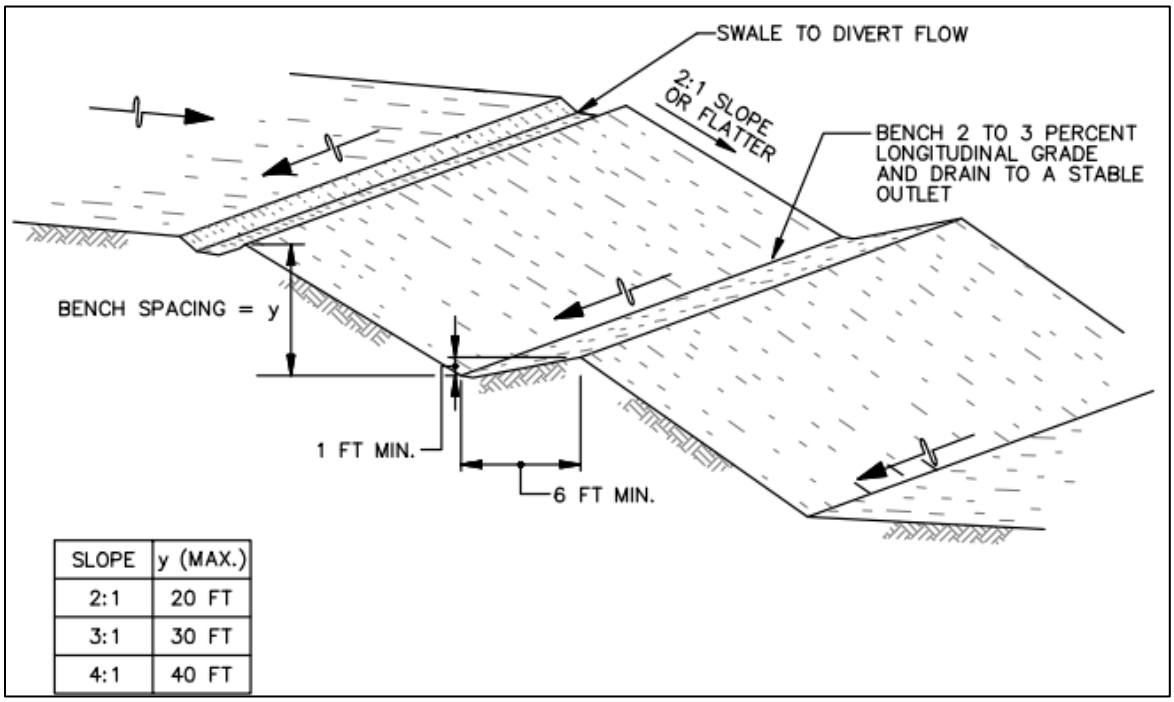


Figure 4.3.1-A: Benching details. Adapted from VDEQ (2024).

Terracing may be the preferred option when adequate space is available, on longer slopes, and when time is a constraining factor. Design features of terracing include (Figure 4.3.1-B):

- Horizontal spacing;
  - Wide enough for a bulldozer (TDOT)
  - At least six feet (MDE 2011)
- Regular breaks in slope;
  - TDOT recommends spacing less than 30 vertical feet for cut slopes or 25 feet for fill slopes
  - MDE (2011) recommends breaks every 20, 30, or 40 vertical feet, based on slope
- Overall slope length;
  - Less than 800 feet (MDE, 2011)
  - Unless other EPSC measures are incorporated

- Slight backslope on the terrace cuts; and
    - Reverse slope of 6H:1V (MDE 2011)
    - Allows runoff to drain toward the original grade, thereby increasing infiltration
- Horizontal, longitudinal slopes channeling runoff to a stable outlet.
    - Two to three percent (MDE, 2011)



SLOPE	y (MAX.)
2:1	20 FT
3:1	30 FT
4:1	40 FT

Figure 4.3.2-B: Terracing details. Source: MDE (2011).

Example Application

No formal design or quantities are required for this measure and, therefore, are not presented here.

References

Hamilton County. (2008). *Stormwater BMP Manual Best Management Practices*.
ITD. (2011). *Best Management Practices Manual*.
MDE. (2011). *Maryland Standards and Specifications for Soil Erosion and Sediment Control*.
Nashville – Davidson County. (2021). *Stormwater Management Manual Vol 4.*
TDOT. Drainage Manual Ch10.
VDEQ. (2024). *Virginia Stormwater Management Handbook*.

4.3.2 Channels and Swales



Source: TDEC

Definition and Purpose

Channels and swales are either natural or constructed wet weather runoff conveyance measures that are shaped or graded to required dimensions per engineering design. When appropriately sized and stabilized (Section 4.2.6.1), channels and swales convey concentrated runoff from the construction site to a receiving channel, waterbody, or other EPSC measure without damage from erosion.

Appropriate Applications

Channels and swales are applicable to construction that produces concentrated runoff. Typical locations include roadside ditches, channels at property boundaries, channels created by diversion structures, or channels and swales designed as part of a permanent storm water conveyance system for the site.

Limitations and Maintenance

Channels and swales require inspections routinely and after heavy rains. Inspect the channel for signs of scour, erosion, and overtopping. These may indicate an under-designed conveyance structure. Additionally, inspect channel and swale outlet(s) for signs of erosion

and sediment deposition. Many outlets need to discharge into outlet protection or an energy dissipator (Section 4.3.4) in order to prevent erosion. If the construction site sediment yield is particularly high, or sediment is regularly observed in the conveyance structure or the outlet, consider implementing additional EPSC measures upstream or in the conveyance structure. Check dams (Section 4.3.3) and sediment tubes or socks (Section 4.4.9) can be used within conveyance structures to enhance runoff velocity control and sediment trapping. Trapped sediment needs to be removed as needed to maintain the design hydraulic capacity.

Planning and Design Considerations

Construct and stabilize channels and swales early in the construction phase; before surfaces are graded or paved, and runoff rates are increased. However, ensure other EPSC measures are in place before disturbance, such that channels and swales do not convey sediment-laden water when used as a diversion. When possible, drainage channels and swales should follow natural contours and make use of existing drainage paths. It is common to place conveyance structures near development edges, along roadways, or at the rear of property lines. Avoid routing them across watershed divides or ridge lines as this can disrupt natural flow patterns and complicate site hydrology. To the greatest extent possible, preserve natural drainageways and incorporate them into the site design (NCDEQ, 2013).

Channels and swale are typically trapezoidal, parabolic, or V-shaped (Figure 4.3.2-A). To select the ideal shape, determine the peak flow rate in which the channel or swale must adequately convey based on CGP or local requirements (Eqn 9 or Eqn 10). Trapezoidal and parabolic channels are ideal for conveying larger flow rates, while V-shaped channels are better for low flows (VDEQ, 2024). Additionally, consider the available space and site specific constraints to determine which shaped cross-section may work best.

Hydraulic computations for each channel shape are governed by channel geometries (Table 2-A), the Continuity Equation (Eqn 7), and Manning's Equation (Eqns 6 and 8). However, tools can be used to reduce computation time, ensure accurate computations, and/or for less typical cross-sectional shapes (Fang, 2007; Mejia & Ponce; Perez et al. 2015) as specified in Section 2.1.2. For adequate hydraulic capacity, ensure the peak flow rate computed from Manning's Equation is greater than the peak flow rate computed from Eqn 9, Eqn 10, or an acceptable modeling software. If stabilization measures are required, be sure the channel or swale is over-excavated to account for the thickness of stabilization measures or that the geometries of the channel or swale account for such thicknesses when computing the hydraulic capacity. Additionally, ensure the proposed side slopes coincide with recommendations for the stabilization measure. Manning's n values can be found for most materials (natural, bioengineered, or structural) in Sections 3.2.1 and 4.2.6; however, if

specific stabilization measures are used, it is best to obtain this value from manufacturer specifications.

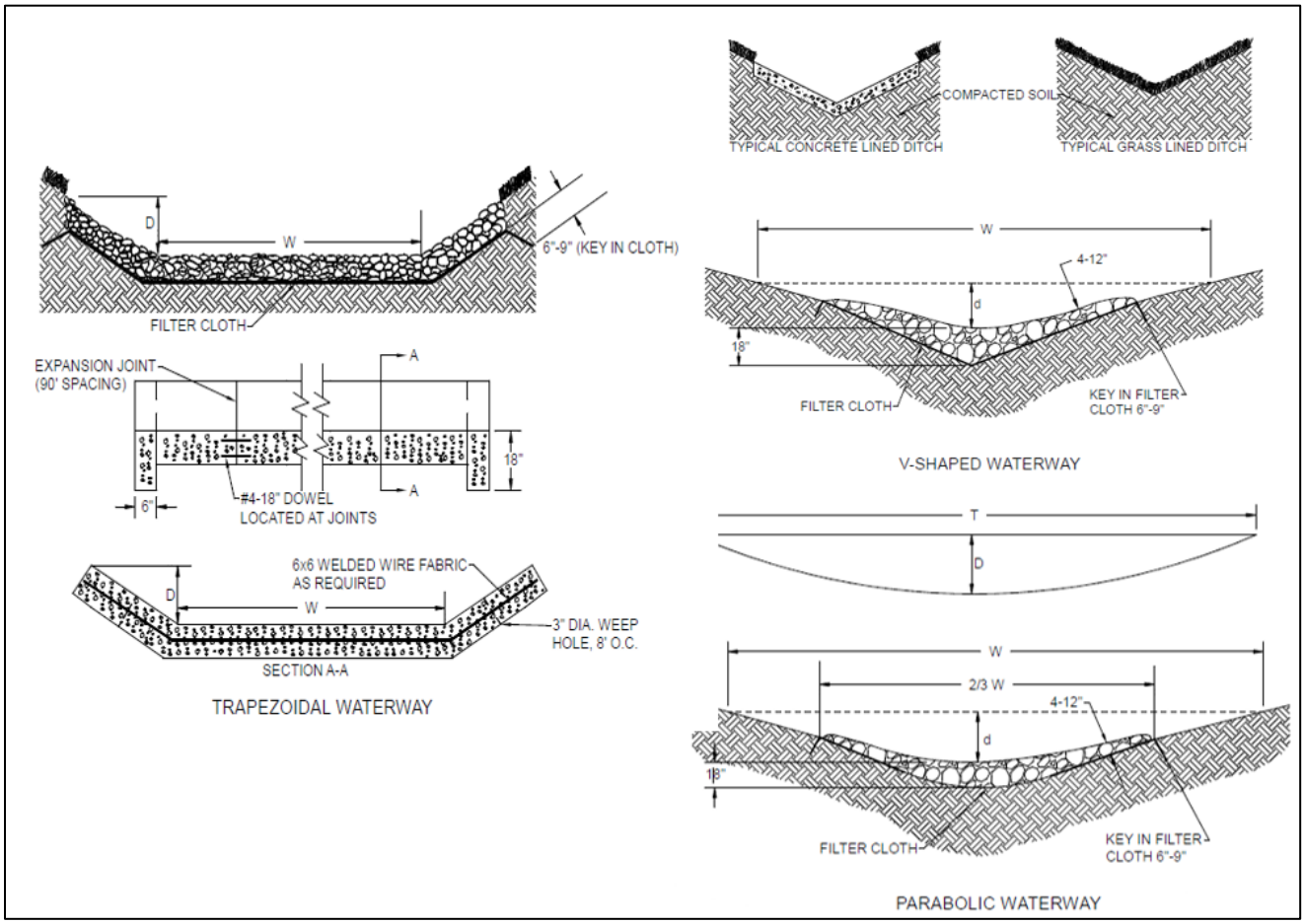


Figure 4.3.2-A: Channel and swale cross-sectional geometries. Adapted from VDEQ (2024).

Design dimensions of channels should be consistent with TDOT methodology. Helpful design charts to eliminate "guess and check" methods or substantially over-sizing the conveyance structure are presented in TDOT as well as Appendix C-ECM-09-a in VDEQ (2024), and Appendix 8.05 in NCDEQ (2013). Use Table 2-A or the previously mentioned tools to make note of the water surface elevation for the cross-section shape of choice during the design flow. Ensure this elevation is greater than the elevation of the receiving water during such a design event to prevent backflow and that this elevation is within the bounds of the channel or swale with sufficient freeboard. Lastly, use the wetted cross-section geometries to calculate the Froude Number, the ratio of internal forces to gravitational forces (Eqn 16):

$$F_r = \frac{V}{(g \times D)^{0.5}} \quad \text{(Eqn 16)}$$

where V is the average cross-sectional design velocity in feet per second, g is the acceleration due to gravity in feet per square second, and D is the hydraulic depth in feet (which can be calculated from Table 2-A or the previously mentioned tools). When the Froude Number is greater than one, less than one, or equal to one, flow in the open channel conveyance is supercritical, subcritical, or critical, respectively. It is ideal to size the channel or swale such that supercritical flow is avoided. Consider altering cross-section geometries to achieve subcritical or critical flow, or retrofit a drop structure and hard armoring in the design.

Again, stabilization of channels and swales is likely necessary when design velocities exceed two feet per second (Section 4.2.6.1). In addition, channel outlets require stabilization. The discharge velocity for the design flow must be nonerosive for the existing receiving system conditions.

General hydraulic design guidance for channels and swales includes:

- Design must consider the construction phase and the permanent stormwater management conveyance (after construction is complete) phase. Factors used in the design should reflect such conditions;
- Where slopes over 10% and/or supercritical flow cannot be avoided, use FHWA's HEC 15 for a more detailed design using hard armoring and/or grade control structures;
- On steep slopes, shorten the effective slope length by installing "turn outs" that discharge runoff non-erosively over stable slopes; and
- At a minimum, the freeboard should be sufficient to prevent waves or fluctuations in the water surface from washing over the sides. In a permanent roadway channel, about half a foot of freeboard is generally adequate, and for transitional channels, zero freeboard may be acceptable. For steep gradient channels, use a freeboard height equal to the flow depth. This allows for large variations to occur in flow depth for steep channels caused by waves, splashing, and surging. Lining materials should extend to the freeboard elevation.

Example Application

Given:

It is anticipated that a construction site will produce a concentrated flow at a design rate of 20 cubic feet per second.

Determine:

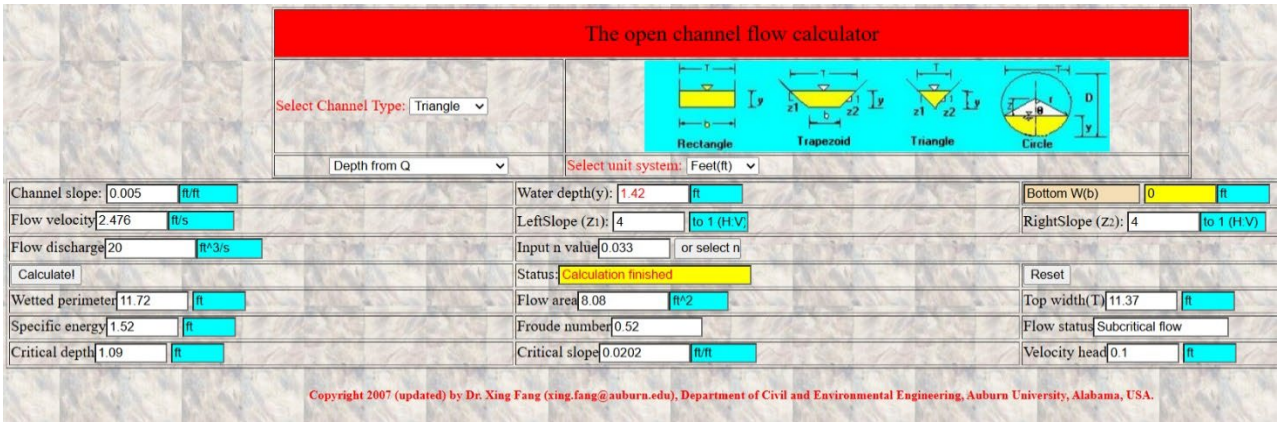
The geometries of a V-shaped channel to safely convey the design flow.

Solution:

Generally, it would be most ideal to refer to the tables and appendices in TDOT, NCDEQ (2013), or VDEQ (2024) to eliminate guess work. However, for sake of demonstration, a run through of the calculations is provided.

Due to the ease of purchase, it has been determined the Class A-1 riprap will be used as a stabilization measure. Class A-1 riprap has a Manning's n value of 0.33 and riprap cannot have a side slope steeper than 2H:1V (Section 4.2.6.4). Space constraints will likely permit side slopes as shallow as 4H:1V at a slope of 0.5%. For an examples using direct application of Manning's Equation refer to Section 4.2.6.1. For this example, "The open channel flow calculator©" was used.

Using this tool, these maximum criteria can be input to the model along with the peak flow rate and it can calculate the hydraulic depth, D, as well as all other hydraulic geometries, as shown below.



These inputs yield a flow depth of 1.42 feet and a suitable design because the flow is subcritical. However, this may not optimally utilize the space on the construction site. Therefore, it may be ideal to input different channel geometries to obtain other suitable designs that utilize less space. With a V-shaped channel, only the side slopes can be adjusted, but for other shapes, the bottom width can also be adjusted. For this example, channel slopes of 3H:1V and 2H:1V were input to the tool. Slopes of 2H:1V produce a V-shaped channel with a flow depth of 1.88 feet that yields subcritical flow. Thus, this design was selected as it utilized the least amount of space. Ensure the channel is excavated to provide sufficient freeboard.

References

Fang, X. (2007). *The open channel flow calculator*. https://www.eng.auburn.edu/~xzf0001/Handbook/Channels.html

Mejia, J. W., & Ponce, V. M. *Geometric elements of a parabolic channel cross section*. https://ponce.sdsu.edu/onlinegeometricelementsparabolic.php

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual.*

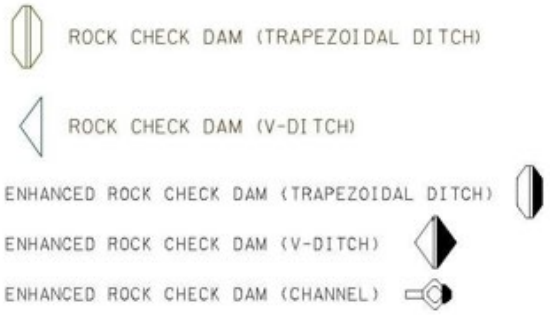
Perez, M. A., Butler, C. G., & Fang, X. (2015). Compute critical and normal depths of arch and elliptical pipes. *Journal of Irrigation and Drainage Engineering*, *141*(9), 06015001.

TDOT. *Drainage Manual Ch5*.

VDEQ. (2024). *Virginia Stormwater Management Handbook*.

4.3.3 Check Dams and Enhanced Check Dams





Source: City of Franklin, TN

Definition and Purpose

Check dams, also known as ditch checks, are small, temporary structures placed across swales, drainage ditches, or other concentrated flow areas, excluding streams, on construction sites. Their primary purpose is to reduce the erosion potential of stormwater by intermittently obstructing the flow and not allowing it to reach its full velocity. These structures are typically constructed from materials like rock, sandbags, or wattles and are most effective when used in combination with other erosion and sediment control practices.

An enhanced rock check dam builds on the design of a traditional check dam, where a layer of smaller mineral aggregate (typically size 57) is placed on the upstream side of the dam and is separated from the original check dam by two layers of geotextile fabric. Like standard check dams, the enhanced version slows the velocity of runoff in drainage ditches, channels, or swales. The finer stones in enhanced check dams promote sediment capture and filtration.

Appropriate Applications

Check dams are best suited in open channels, such as swales, roadside ditches, and temporary drainageways. They are a practical solution where erosion and sedimentation problems exist in areas of concentrated flow and permanent stabilization is not feasible due to timing or the nature of the channel. This includes situations where grass or vegetative

linings are not taking root or where construction or weather delays prevent immediate installation of other stabilization measures.

Limitations and Maintenance

Check dams are to be used in conjunction with other EPSC measures, are not considered a primary sediment control measure, and are not to be used in streams (MPCA, 2023). Check dams become less effective in channels exceeding 10% in slope and when the contributing drainage area exceeds 10 acres (TDOT). Enhanced check dams become less effective when drainage areas exceed 20 or 30 acres, depending on whether the channel drains to an ETW/water with unavailable parameters or a water with available parameters, respectively (TDOT). For larger drainage areas, consider gabion check dams (Section 4.3.3.1), diverting unpolluted runoff away from the main channel (Section 4.2.3), or a greater number of dams.

Because check dams pond water to slow the velocity, they can inadvertently hinder vegetation by smoothing and smothering existing grasses and preventing new vegetation from taking hold. They can also disrupt the natural flow of water, which may reduce channel capacity and cause downstream turbulence. Therefore, inspections of check dams are needed after heavy rainfall and on a regular basis. If downstream turbulence (i.e., supercritical flow; Eqn 16 [Section 4.3.2]) and erosion are noted, additional runoff control and erosion prevention measures are likely necessary downstream (ALSWCC, 2018; NCDEQ, 2013). During runoff-producing events, ensure stormwater passes over the center of each dam and does not excessively penetrate or undercut the dam (i.e., it is impossible to completely eliminate penetration and undercutting) or cut around the sides of the dams. Additionally, ensure the center sag of the dam is at a height such that the dam effectively ponds water and does not immediately pass the runoff. If excessive penetration, undercutting, or passing occurs, the dams likely need to be reshaped. Though not considered a primary sediment control measure, make note of accumulated sediment behind the dams, particularly sediment accumulation behind the mineral aggregate of enhanced check dams. Remove the accumulated sediment before or once it reaches 50% capacity. Check dams should be kept in place until permanent stabilization is established, after which, they must be removed unless they are biodegradable and the area in which they were located is re-stabilized.

Planning and Design Considerations

The channel (Section 4.3.2) and check dam must be designed to adequately convey the design storm for the associated drainage area. Check dams are most commonly constructed with conventional riprap, but other materials such as sandbags, gravel-filled bags, fiber logs, tubes, or wattles (Section 4.4.9), and enhanced silt fence (see appropriate standard drawings, Section 4.4.10 only details enhanced silt fence used in sheet flow applications) can also be

used (USEPA, 2021). When constructing check dams with materials such as fiber logs, tubes, or wattles, follow the manufacturer's instructions regarding product applications, limitations, and installation. Many of these products require staking, stapling, or trenching as well as overlapping abutments. Use longer stakes and reduce the spacing where higher velocities are anticipated. Do not stake through the product as this creates a preferential flow path (i.e., an area for water to more easily flow through) and may lead to flotation of the product. Follow specifications in the appropriate standard drawings for enhanced silt fence. For products not requiring anchoring, such as sandbags, it is best to lay the product in a staggering, brick-like pattern. More detailed installation specifications for wattles are also provided in Section 4.4.9. Materials such as straw bales and silt fences are not suitable check dam materials as they are not manufactured to withstand concentrated flows (MPCA, 2019).


Source: TNWRRC

It is ideal to use check dams in series. This practice does not allow runoff to regain velocity as it traverses down gradient and further promotes the settling of sediments. However, if sediment is continually observed upgradient of the dam(s), designers are encouraged to incorporate (additional) measures into the overall EPSC plan (USEPA, 2021).

Though a formal design of check dams is not required, there are various design considerations that promote long-term and effective function of check dams. Ultimately, it is the responsibility of the designer to ensure that a check dam system is fully functional. Best practices to ensure check dams are functional include:

- The depth of flow over the center of the structure (weir) should adequately convey the design storm without flow overtopping the channel;
- Six to 12 inches of height from the top of the weir to the top of the channel. (ALSWCC, 2018; NCDEQ, 2013; VDEQ, 2024);
- The horizontal spacing of dams is dependent on channel slope and weir height. Set the spacing such that the elevation at the top of the weir of the downstream dam is the same as the toe elevation of the upstream dam (Figure 4.3.3-A, B);

- Shallow side slopes of the dams. Most sources suggest 2H:1V or flatter (ALSWCC, 2018; MPCA, 2023; NCDEQ, 2013; VDEQ, 2024), while TDOT suggests a more conservative 3H:1V side slope; and
- The use of a geotextile filter fabric under the base of the check dams that extends three feet beyond the toe of the dam (TDOT, VDEQ, 2024). This fabric is particularly helpful for stone dams and will act as a separator between the dam and soil base abutments. The fabric is to be set in the subgrade soils, which will help prevent soil particle migration, scour, and undercutting.

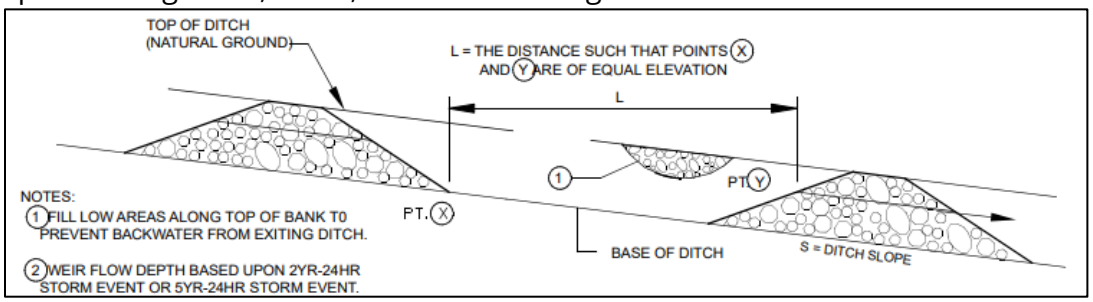


Figure 4.3.3-A: Spacing of check dams.

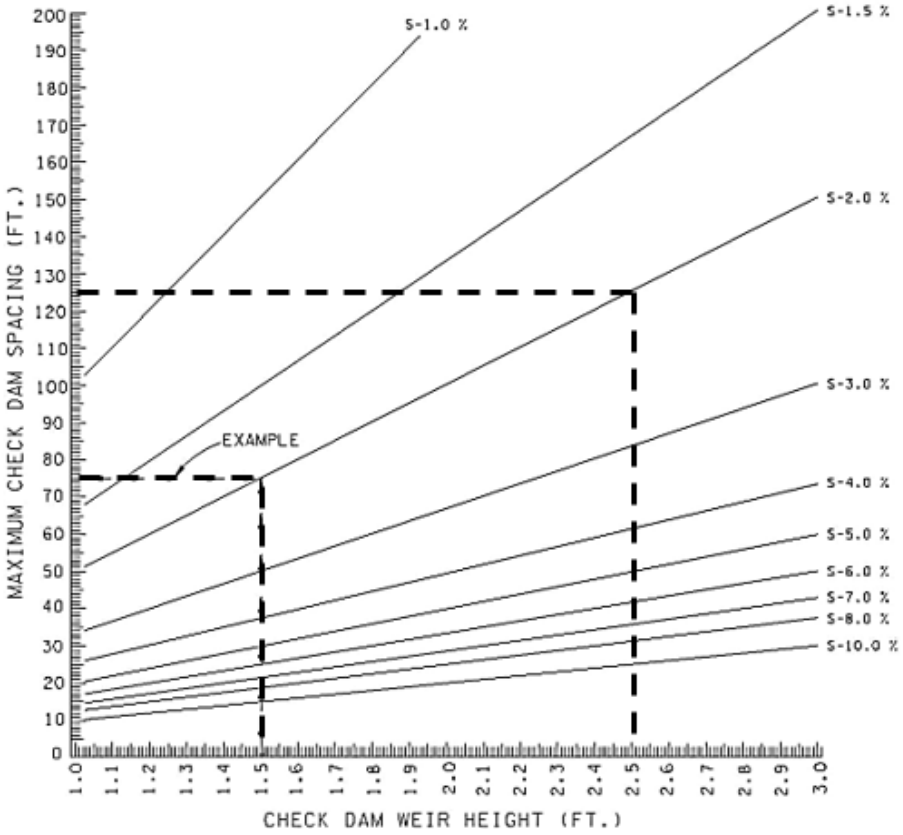


Figure 4.3.3-B: Maximum spacing of check dams governed by channel slope. Source: TDOT

All planning and design considerations of traditional check dams are applicable to enhanced check dams. However, formal design of enhanced check dams is required when draining an area of 20 acres or more (TDOT). Like traditional check dams, the weir of enhanced check dams should not be more than three feet above the channel bottom and should be between half a foot and one foot below the channel banks. Because drainage areas may be larger for enhanced check dams, the weir height below the channel banks should be maximized whenever possible (i.e., as close to one foot below the channel banks). This will maximize the filtration capacity of the structure (TDOT). However, when the drainage area of the channel exceeds 20 acres, it is recommended that the designer compute the depth of runoff over the weir for the design storm to ensure the depth of runoff does not overtop the weir. This can be checked with standard weir equations. Flow over check dam weirs is most commonly governed by rectangular (Eqn 17) or trapezoidal (Eqn 18) weir equations:


Source: TNWRRC

$$Q = C \times L \times H^{1.5} \quad \text{(Eqn 17)}$$

$$Q = \frac{2}{5} \times C \times z \times H^{2.5} \quad \text{(Eqn 18)}$$

where C is a unitless weir coefficient (usually 3.33 for sharp crested or 3.09 for broad crested weirs), L is the length of the weir in feet, H is the height of water above the weir in feet, and z is the side slope in feet per foot (H:V). In general, software such as HydroCAD® may be helpful to accurately compute more complex scenarios. When the flow over the weir exceeds one foot, and the depth of the ditch is less than two feet, the designer is encouraged to consider other EPSC measures, since it will not be possible to provide the minimum weir height of one foot above the channel bottom.

Under certain circumstances, enhanced rock check dams may remain in place as a permanent ditch check at locations where removal would be difficult and might cause significant disturbance. If known during EPSC plan preparation, the designer should indicate on the plans which (if any) should remain in place.

Example Application

-Example courtesy of TDOT-

Given:

A proposed V-shaped ditch with 6H:1V side slopes has been designed along a roadway. The proposed ditch will be 200 feet long with a grade of 2% and a depth of 3.5 feet.

Determine:

The number of check dams, sizing, and spacing.

Solution:

Step 1 – Determine the height of the weir: Since the ditch is 3.5 feet deep, an ideal height of the weir may be 2.5 feet, such that it is less than 3 feet above the channel bottom and 1 foot below the channel sides.

Step 2 – Determine the spacing of the check dams: The maximum dam spacing can be calculated by dividing the dam height by the channel grade.

Spacing (ft) = dam height (ft) / channel grade = 2.5 / 0.02 = 125 ft

The spacing can also be determined by using Figure 4.3.3-B. This figure also yields an answer of 125 feet.

Step 3 – Determine the number of check dams: The number can be computed by dividing the channel length by the spacing and rounding up to the next whole number.

Number of dams = Length / spacing = 200 / 125 = 1.6 = 2 dams

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.
MPCA. (2023). *Minnesota Stormwater Manual: Sediment control practices – Check dams.*
NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual.*
TDOT. *Drainage Manual Ch8*.
TDOT. *Drainage Manual Ch10*.
USEPA. (2021). *Stormwater Best Management Practices: Check Dams*.
VDEQ. (2024). *Virginia Stormwater Management Handbook*.

4.3.3.1 Gabion Check Dams





GABION CHECK DAM

Source: Environmental Geological Agency

Definition and Purpose

Gabion check dams are check dams constructed from gabion baskets and placed across a swale, drainage ditch, or other concentrated flow areas, excluding streams. These check dams consist of two rows; one placed on top of the other, with the upper center basket removed to act as the weir. Their primary purpose is to reduce the velocity of stormwater by flattening the slope of the channel, thereby reducing erosion.

Appropriate Applications

Gabion check dams are generally applicable where traditional check dams are used (Section 4.3.3). However, gabion check dams may be preferable over traditional check dams where dam stability is of concern. These scenarios may include where (TDOT):

- Channel drainage areas exceed 10 acres;
- Channel slopes exceed 4%;
- Site conditions restrict or limit equipment access, which impose challenges of creating check dams through dumping rocks or other materials; and
- Permanent runoff velocity control is desired.

Limitations and Maintenance

All types of check dams are not to be used in streams. The construction of gabion check dams requires more time and will likely be more costly than traditional check dams and other potential EPSC measures. Furthermore, if gabion check dams are used as a temporary measure, the removal of such structures results in more disruption to the channel slopes

and stability. This is because excavation will be required to remove riprap wedges, coarse aggregate used to fill void spaces between the gabions and natural ground, and gabions built into the channel sidewalls. Gabion check dams are less functional when (TDOT):

- Channel drainage areas exceed 35 acres;
- Channels are not properly stabilized, are subjected to larger shear forces and velocities, and drain highly erodible watersheds;
- Channel slope exceeds 10%;
- Channel side slopes are steeper than 2H:1V;
- Channels require vegetation establishment; and
- Other scenarios are present that may indicate excessive sediment yields.

In addition to inspection points for traditional check dams, the wire baskets must be inspected on gabion check dams. Inspect the wire baskets for signs of abrasion and wear.

Planning and Design Considerations

Similar to traditional check dams, a Type III geotextile fabric is to be installed between the base of the gabion structure and the channel bed and extend three feet beyond the toe of the riprap slope. Because gabion baskets are pre-fabricated at standard sizes, the remaining planning and design considerations differ from traditional check dam design. Nonetheless, formal design is only required when gabion check dams are implemented as permanent EPSC measures. Each gabion check dam used should be labeled on the EPSC plans as permanent or temporary (or both) with corresponding information such as the required weir width, the size and length of gabion for the lower row, and the size and length of gabion for the upper row.

A variety of gabion check dam weir sizes and configurations may be applicable to convey the peak flow rate from the design storm (Table 4.3.3.1-A). These provided flow rates were calculated to provide ample protection before channel stability or lining measures have been implemented, and are much lower than the necessary flow rates to overtop the structure. Thus, when left in place as permanent structures, gabion check dams sized in accordance with Table 4.3.3.1-A will be able to convey peak flow rates from larger storms such that the channel stability measures were implemented with good practices (Section 4.2.6).

Table 4.3.3.1-A: Flow rate capacities for various gabion configurations, Source: TDOT.

18-inch Upper Gabion Row

Bottom Width of Channel (ft)	12-inch Lower Gabion						18-inch Lower Gabion					
	Weir Length (ft)			Allowable Flow Rate (cfs)			Weir Length (ft)			Allowable Flow Rate (cfs)		
	Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)		
	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1
3	2.6	3.3	7.6	13.4	18.8	13.4	8.6	12.3	19.6	23.1	31	35.6
4	3.6	4.3	8.6	13.2	17.7	14.7	9.6	13.3	20.6	25.9	33.8	38.5
5	4.6	5.3	9.6	13.9	17.9	16	10.6	14.3	21.6	28.8	36.7	41.4
6	5.6	6.3	10.6	15	18.7	17.4	11.6	15.3	22.6	31.7	39.6	44.4
7	6.6	7.3	11.6	16.1	19.9	18.7	12.6	16.3	23.6	34.7	42.6	47.2
8	7.6	8.3	12.6	17.5	20.9	20.1	13.6	17.3	24.6	37.7	45.7	50
9	8.6	9.3	13.6	18.8	22.2	21.5	14.6	18.3	25.6	40.7	48.4	53.2
10	9.6	10.3	14.6	20.3	23.4	23	15.6	19.3	26.6	43.8	51.6	56.2
12	11.6	12.3	16.6	23.2	26.1	26	17.6	21.3	28.6	49.8	57.7	62.3
15	14.6	15.3	19.6	27.8	30.5	30.5	20.6	24.3	31.6	59.2	66.9	71.4

12-inch Upper Gabion Row

Bottom Width of Channel (ft)	12-inch Lower Gabion						18-inch Lower Gabion						36-inch Lower Gabion					
	Weir Length (ft)			Allowable Flow Rate (cfs)			Weir Length (ft)			Allowable Flow Rate (cfs)			Weir Length (ft)			Allowable Flow Rate (cfs)		
	Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)		
	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1
3	4.2	4.5	6	4.7	6.3	6.9	6.2	7.5	10	7.6	9.9	11.4	12.2	16.5	22	20.4	27.8	34.3
4	5.2	5.5	7	5.6	7.1	7.7	7.2	8.5	11	9	11.3	12.8	13.2	17.5	23	23.2	30.6	37.1
5	6.2	6.5	8	6.5	7.9	8.6	8.2	9.5	12	10.4	12.7	14.3	14.2	18.5	24	26.1	33.6	40
6	7.2	7.5	9	7.5	8.8	9.5	9.2	10.5	13	11.9	14.2	15.7	15.2	19.5	25	29	36.5	42.9
7	8.2	8.5	10	8.4	9.8	10.4	10.2	11.5	14	13.4	15.7	17.2	16.2	20.5	26	31.9	39.5	45.9
8	9.2	9.5	11	9.4	10.7	11.4	11.2	12.5	15	14.9	17.1	18.6	17.2	21.5	27	24.9	42.5	48.8
9	10.2	10.5	12	10.4	11.7	12.3	12.2	13.5	16	16.4	18.6	20.1	18.2	22.5	28	37.9	45.3	51.8
10	11.2	11.5	13	11.4	12.6	13.3	13.2	14.5	17	18	20.1	21.6	19.2	23.5	29	41	48.3	54.8
12	13.2	13.5	15	13.4	14.6	15.2	15.2	16.5	19	21	23.2	24.6	21.2	25.5	31	47.1	54.4	60.9
15	16.2	16.5	18	16.5	17.6	18.2	18.2	19.5	22	25.6	27.8	29.2	24.2	28.5	34	56.4	63.6	70.1

When gabion check dams are intended to be permanent structures, two checks are required:

1. Engineering computations to ensure that the gabion structure will not be overtopped; and
2. Engineering computations to ensure that the gabion structure can withstand the shear forces it is subject to.

The depth of flow over the weir can be computed utilizing standard weir equations (Eqn 17 or Eqn 18 from Section 4.3.3) and solving for height.

Because the wire mesh of gabion structures allows the structure to act as a single unit (Section 3.3.2.1), the gradation of stone used is less critical for shear stress calculations. However, the gradation is important so that void spaces in the gabion structure are minimized, such that runoff cannot easily flow through the dam. TDOT suggests a gradation of stone smaller than Class A-1. Thus, the allowable shear stress is more dependent on the wire. A typical allowable shear stress is that of 5.3 pounds per square foot, although it is more ideal to check manufacturer specifications, if such a threshold is provided.

By definition, the critical depth is the depth of flow over the weir that maximizes the discharge at a given specific energy or minimizes the specific energy for a given discharge. Although the water surface would be at the depth, H, by Eqn 17 (rectangular weir), as it flows onto the weir, it will pass through critical depth, d_c, as it flows across the downstream side of the weir. Since this is the minimum depth which will occur on the weir, this also represents the point of greatest velocity and shear stress. The equation for calculating shear stress at critical depth is similar to Eqn 12 expressed in Section 3.3.2.1:

$$\tau_c = \gamma \times d_c \times S_c \qquad \text{(Eqn 19)}$$

where γ is the unit weight of water (62.4 pounds per cubic foot), d_c is the critical depth of flow in feet, and S_c is the critical slope (the slope at which normal depth and critical depth are equivalent) expressed in feet per foot. Because gabion baskets are rectangular prisms, only rectangular weirs can be constructed and the critical depth for rectangular cross-section weirs is calculated by:

$$d_c = \left(\frac{Q^2}{g \times L^2}\right)^{1/3} \qquad \text{(Eqn 20)}$$

where Q is discharge in cubic feet per second, L is the length of the weir in feet, and g is the acceleration due to gravity (32.2 feet per square second). Critical slope for rectangular weirs can be calculated by rearranging Manning's Equation:

$$S_c = \left(\frac{Q \times n \times (L + 2 \times d_c)^{2/3}}{1.486 \times (L \times d_c)^{5/3}}\right)^2 \qquad \text{(Eqn 21)}$$

where n is Manning's roughness coefficient, which is typically 0.069 for gabion stone filler. Be sure to check the roughness coefficient from the supplier or Table 4.2.6.4-A.

Similar to weir sizing, a variety of gabion sizes may be applicable for the dam and are to be spaced apart according to Table 4.3.3.1-B. Selecting the sizes of the lower and upper gabion rows is to be in accordance with Tables 4.3.3.1-C and 4.3.3.1-D, respectively. When multiple sizing options are available for the lower gabion row, selecting the taller configuration is generally recommended. For instance, if a channel with 3H:1V side slopes is designed to convey a flow of 13 cubic feet per second, the lower gabion row could be either 12 or 18 inches tall (Table 4.3.3.1-B). In this scenario, an 18-inch height is preferred, as long as it does not cause the overall structure to extend above the channel banks. That is, the sizing of the upper gabion following guidance in Table 4.3.3.1-C but must not exceed the height of the channel banks once stacked on the bottom row. Lastly, refer to Table 4.3.3.1-D for the spacing of multiple gabion check dams within a channel.

Table 4.3.3.1-B: Maximum spacing for gabion check dams. Source: TDOT.

Ground Slope (ft/ft)	Maximum Spacing (ft)		
	12-inch Basket	18-inch Basket	36-inch Basket
0.01	72	122	272
0.015	47	81	181
0.02	35	60	135
0.03	22	39	89
0.04	16	29	66
0.05	12	22	52
0.06	10	18	43
0.07	-	15	37
0.08	-	13	32
0.09	-	11	28
0.1	-	10	25
0.11	-	-	22
0.12	-	-	20
0.13	-	-	19
0.14	-	-	17
0.15	-	-	16
0.2	-	-	11

Table 4.3.3.1-C: Design dimensions for lower gabion rows. Source: TDOT.

Bottom Width of Channel (ft)	12-inch Lower Gabion									18-inch Lower Gabion									36-inch Lower Gabion								
	Min. Gabion Length (ft)			Lower Embedment Length (ft)			Upper Embedment Length* (ft)			Min. Gabion Length (ft)			Lower Embedment Length (ft)			Upper Embedment Length* (ft)			Min. Gabion Length (ft)			Lower Embedment Length (ft)			Upper Embedment Length* (ft)		
	Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)		
	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1
3	9	9	12	3	3	4.5	1.5	0.75	1.5	9	12	15	3	4.5	6	0.5	0.75	1	15	21	27	6	9	12	0.5	0.75	1
4	9	9	12	2.5	2.5	4	1	0.25	1	12	15	15	4	5.5	5.5	1.5	1.75	0.5	18	21	27	7	8.5	11.5	1.5	0.25	0.5
5	9	12	12	2	3.5	3.5	0.5	1.25	0.5	12	15	18	3.5	5	6.5	1	1.25	1.5	18	24	30	6.5	9.5	12.5	1	1.25	1.5
6	12	12	15	3	3	4.5	1.5	0.75	1.5	12	15	18	3	4.5	6	0.5	0.75	1	18	24	30	6	9	12	0.5	0.75	1
7	12	15	15	2.5	4	4	1	1.75	1	15	15	18	4	4	5.5	1.5	0.25	0.5	21	24	30	7	8.5	11.5	1.5	0.25	0.5
8	12	15	15	2	3.5	3.5	0.5	1.25	0.5	15	18	21	3.5	5	6.5	1	1.25	1.5	21	27	33	6.5	9.5	12.5	1	1.25	1.5
9	15	15	18	3	3	4.5	1.5	0.75	1.5	15	18	21	3	4.5	6	0.5	0.75	1	21	27	33	6	9	12	0.5	0.75	1
10	15	15	18	2.5	2.5	4	1	0.25	1	18	18	21	4	4	5.5	1.5	0.25	0.5	24	27	33	7	8.5	11.5	1.5	0.25	0.5
12	18	18	21	3	3	4.5	1.5	0.75	1.5	18	21	24	3	4.5	6	0.5	0.75	1	24	30	36	6	9	12	0.5	0.75	1
15	21	21	24	3	3	4.5	1.5	0.75	1.5	21	24	27	3	4.5	6	0.5	0.75	1	27	33	39	6	9	12	0.5	0.75	1

*Assuming 3-inch embedment depth at base of gabion

Table 4.3.3.1-D: Design dimensions for upper gabion rows. Source: TDOT.

Bottom Width of Channel (ft)	12-inch Upper Gabion									18-inch Upper Gabion								
	Min. Gabion Length (ft)			Lower Embedment Length (ft)			Upper Embedment Length* (ft)			Min. Gabion Length (ft)			Lower Embedment Length (ft)			Upper Embedment Length* (ft)		
	Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)			Side Slope (H:V)		
	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1	2:1	3:1	4:1
3	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3
4	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3
5	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3
6	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3
7	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3
8	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3
9	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3
10	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3
12	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3
15	3	6	6	2.1	4.5	4.5	0.1	1.5	0.5	6	9	9	3.3	5.4	6.3	0.3	0.9	0.3

Example Application

-Example Courtesy of TDOT-

Given:

A roadway side ditch for a pavement replacement project on a freeway will direct the runoff from approximately 5 acres into a proposed swale, which will carry flows to a river that runs parallel to the roadway. The roadway is situated at the top of a hill that slopes steeply down to the river, which has been listed as an ETW. The proposed swale will have a planimetric length of 170 feet and will be at a slope of 8.0%. It is proposed to have a 4-foot bottom width with 3H:1V side slopes. The height of this swale, from the flow line to the top of the bank, is 4 feet. Due to the environmentally sensitive nature of the receiving waters, it is desired to provide a vegetated lining for the swale, if possible. Rational Method computations have determined that the 5-year peak discharge will be 22.2 cfs while the 50-year peak discharge will be 29.2 cfs.

Determine:

The required gabion check dam configurations (weir length, gabion sizing, and dam spacing).

Solution:

Step 1 - Determine the required gabion check dam configuration: Since this site drains to an ETW, the flow rates provided in Table 4.3.3.1-A will be compared with the 5-year peak flow rate at the site in order to select a suitable gabion check dam configuration. This table contains one row, which corresponds to a channel bottom width of 4 feet; and on that row, there are five possible configurations that correspond to 3H:1V channel side slopes. However, the allowable flow rates for three of these configurations are less than the 5-year flow rate at the site; thus, there are two possible configurations:

- Option 1 consists of a 36-inch lower gabion row, with a 12-inch upper gabion row, a weir width of 17.5 feet, and an allowable flow rate of 30.6 cfs; and
- Option 2 consists of an 18-inch lower gabion row, with an 18-inch upper gabion row, a weir width of 12.3 feet, and an allowable flow rate of 33.8 cfs.

It is decided to use Option 1 as it will fit within the proposed 4-foot depth of the swale, and the lower gabion is larger, which is preferred.

Tables 4.3.3.1-C and 4.3.3.1-D specify the required lengths of gabion in each row and the lengths of embedment into the sides of the channel for the ends of the two rows of gabions. Based on the table for the lower row, the lower row will have a total length of 21 feet, with an upper embedment length of 0.25 feet and a lower embedment length of 8.5 feet. Based on the table for the upper row, the upper row will require a gabion length of 6 feet on each side of the structure, as well as an upper embedment length of 1.5 feet and a lower embedment length of 4.5 feet (Table 4.3.3.1-D).

It is also useful to determine the depth of embedment of the lower gabion row into the channel bottom. The downstream face of the lower row should be embedded 3 inches. Since the proposed swale will have a slope of 8.0%, the depth of embedment on the upstream side of the lower gabion row may be computed as:

$$\text{Depth of Embedment} = 3\text{in} + (36\text{in} \times 0.08) = 5.9 \text{ inches}$$

The planimetric length of the riprap wedge should also be determined. The 2H:1V slope of the wedge corresponds to a 50% slope. Since the ditch will be at an 8.0% slope, the difference between these corresponds to a slope of 42%. The height of the upstream end of the riprap wedge will correspond to the height of the lower gabion row, which, accounting for the 3-inch embedment depth, is 35 inches or 2.92 feet. Thus, the length of the wedge will be:

$$\text{Length of wedge} = 2.92 \text{ ft} / 0.42 = 6.95 \text{ ft}$$

Step 2 - Check the weir width for the 50-year discharge: Since the gabion check dams are to be left in place as permanent structures, the adequacy of the proposed weir width should be checked for the 50-year peak discharge. As discussed in the Planning and Design Criteria, this involves checking both the depth of flow over the weir (Eqn 17) as well as the shear stress at critical depth (Eqn 19).

$$Q = C \times L \times H^{1.5}$$

$$29.2 = 3.09 \times 17.5 \times H^{1.5}$$

$$H = \left(\frac{29.2}{3.09 \times \mathbf{17.5}}\right)^{2/3} = 0.66 \text{ ft}$$

Since the computed depth of flow is less than the 1-foot upper gabion height, the proposed weir passes the first check. The second check involves determining the shear stress at critical depth. To determine the shear stress, the critical depth and critical slope must first be computed.

$$d_c = \left(\frac{Q^2}{g \times L^2}\right)^{1/3} = \left(\frac{29.2^2}{32.2 \times 17^2}\right)^{1/3} = 0.44 \text{ ft}$$

$$S_c = \left(\frac{Q \times n \times (L+2 \times d_c)^{2/3}}{1.486 \times (L \times d_c)^{5/3}}\right)^2 = \left(\frac{29.2 \times 0.069 \times (17.5+2 \times 0.44)^{2/3}}{1.486 \times (17.5 \times 0.44)^{5/3}}\right)^2 = 0.1 \text{ ft/ft}$$

Finally, the shear stress may be computed from Eqn 19:

$$\tau_c = \gamma \times d_c \times S_c = 62.4 \times 0.44 \times 0.1 = 2.75 \text{ lb/ft}^2$$

Since the computed shear is less than the allowable shear of 5.3 lb/ft^2, the proposed weir passes both tests for the 50-year peak discharge.

Step 3 - Determine the check dam spacing: Since the slope of the proposed swale is 8.0%, the check dam spacing may be determined directly from Table 4.3.3.1-B. Once the spacing has been determined, it is possible to determine the number of check dams required for the 170-foot length of the proposed swale. However, two aspects of this computation should be noted first. One is that the spacing provided on the table does not represent the length of the gabion basket. Thus, the total spacing used to compute the number of dams should therefore be based on the distance provided from the table (32 feet) plus the width of the lower gabion row (36 inches), or 35 feet total. The second aspect is that the most downstream check dam will be located at the end of the proposed swale. Given the lengths of the riprap wedge and lower gabion row, the upstream face of this check dam will be 9.5 feet upstream of the swale outfall. Since the remaining balance of the swale length is 160.5 feet, the number of check dams should be computed as:

Number of dams = 160.5 / 35 = 4.6 = 5 dams

As determined above, the total length of each check dam, including the riprap wedge, will be 9.5 feet. Thus, for 5 dams, the total length of hard revetment will be 47.5 feet, which is between one-fourth and one-third of the total swale length of 170 feet. This runs somewhat counter to the initially stated goal of providing a grassed swale for

flows to the receiving stream. Thus, using procedures described in Section 4.2.6.1, the possibility of using turf reinforcement mats to provide a permanent lining for the swale can be investigated. Although the design computations are not presented here, it is found that a Class II TRM would provide an adequate lining for the 50-year event. Further, the shear stress on the unvegetated liner for the peak 5-year discharge would be 2.98 lb/ft^2, so that an additional riprap lining on the bottom of the swale would not be required to provide erosion protection before the establishment of vegetation in the lining. In the final analysis, the selection of erosion prevention measures for this site should be based on economic considerations.

References

TDOT. *Drainage Manual Ch10*.

4.3.4 Energy Dissipator



Source: TDOT

Definition and Purpose

Energy dissipators are components of drainage structures that are used to prevent erosion or scouring at the outlet of a pipe or culvert. They are often referred to as "integrated" components of drainage structures, as they are not standalone measures. Components may include roughness elements, baffles, broken-back culvert modifications, weirs, impact basins, and drop structures. These work to dissipate flow energy and reduce erosion potential by creating hydraulic jumps, dispersing flow area, and/or disrupting steep slopes.

Appropriate Applications

An energy dissipator should be considered for use where:

- The required length or dimensions of "external" structural energy dissipators such as level spreaders (Section 4.3.5), riprap aprons (Section 4.3.6), or stilling basins (Section 4.3.7) would be excessive or infeasible due to culvert outlet conditions;
- Culvert outlet velocities and shear stresses exceed erosion resistance thresholds; or
- Erosion at the culvert outlet will pose an unacceptable risk to adjacent or downstream property.

Energy dissipators can be used in series when one may not fully prevent scouring or erosive forces, or in combination with structural energy dissipators. A common dual application is implementing a drop structure followed by a weir.

Limitations and Maintenance

An energy dissipator may not be necessary at sites where the natural stream slope is steep, resulting in a high flow velocity in the natural channel. Furthermore, energy dissipators will generally not be required at sites where a channel is lined with bedrock. Energy dissipators require routine inspections to ensure they are functioning properly, which may include inspections during and after storm events. Report, correct, and/or replace any malfunctions promptly. Undermining of the concrete apron or slab may occur when transitions from the pipe to the energy dissipater are not seamless. That is, flowing water finds a seam and goes under the concrete slab or other structure, similar to piping. In these cases, water can erode the base stone and cause a structural failure.

Planning and Design Considerations

There are various energy dissipators that can be implemented in drainage (Figure 4.3.4-A). The following summaries provide an understanding of the benefits and drawbacks of such devices and implementation strategies.

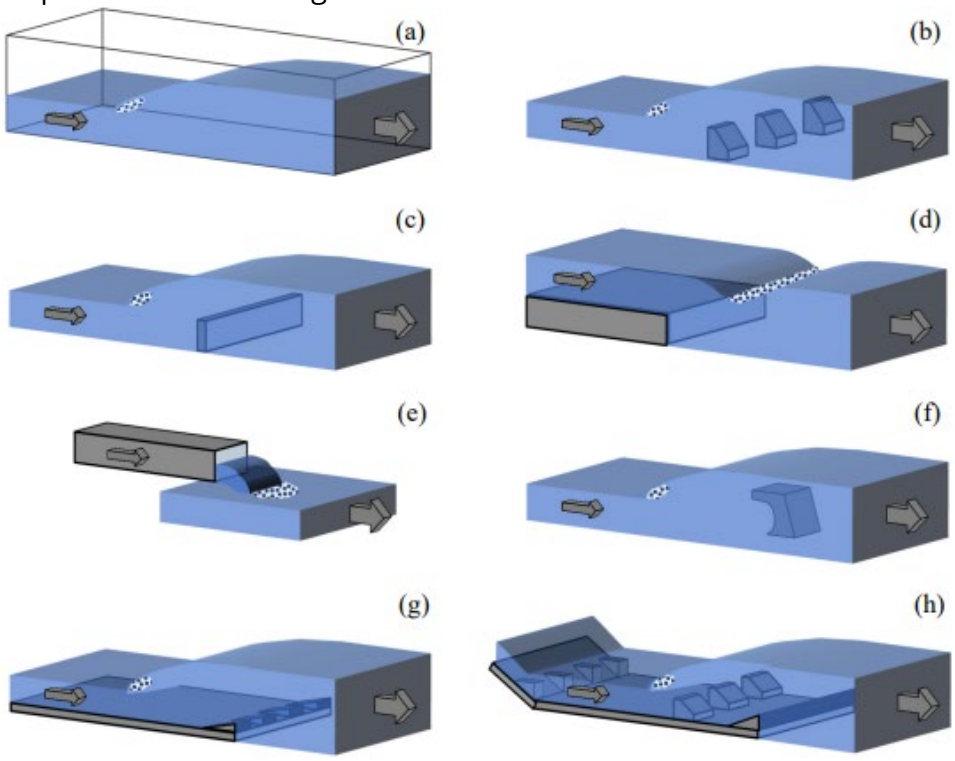


Figure 4.3.4-A: Energy dissipators (a) free hydraulic jump; (b) baffles; (c) sills (weir); (d) drops; (e) plunge basin; (f) counterflow mechanism; (g) flip buckets; and (h) design basins. Source: Admiraal and Zhang (2023).

Roughness elements or increased resistance is a practical method for reducing kinetic energy in flowing water by enhancing hydraulic resistance. This technique involves the strategic placement of roughness elements like protrusions along the culvert to reduce flow velocities by inducing tumbling flow or increased resistance flow. The distinction between these two flow behaviors is critical in design: steep slopes favor tumbling flow, while flatter slopes benefit more from increased resistance. Tumbling flow consists of repeated hydraulic jumps and overfalls that maintain flow at approximately critical velocity, even in conditions where supercritical velocities would otherwise dominate. While not completely eliminating the need for outlet protection, increased roughness can reduce outlet velocities and Froude numbers enough to allow for simpler, lower-cost measures at the outlet. However, these systems are not suitable for channels expected to carry large debris or coarse sediment such as cobbles and boulders. Proper sizing of the culvert and roughness elements, as well as consideration of flow control (inlet versus outlet control), are essential to achieving effective velocity reduction without compromising culvert capacity. The design of tumbling flow systems must account for the relationship between element height and unit discharge, which can limit their use in high-flow situations. Practical implementation often involves five (or more) rows of roughness elements to establish a cyclical flow pattern without needing to line the entire culvert length (Figure 4.3.4-B). Proper spacing, typically placing the first and last elements a distance of half their length from their respective starting points, is essential for maintaining uniform flow. Design success depends on verifying inlet-controlled drainage structures, evaluating outlet velocity needs, calculating critical flow conditions, and appropriately sizing the elements (FHWA, 2006).

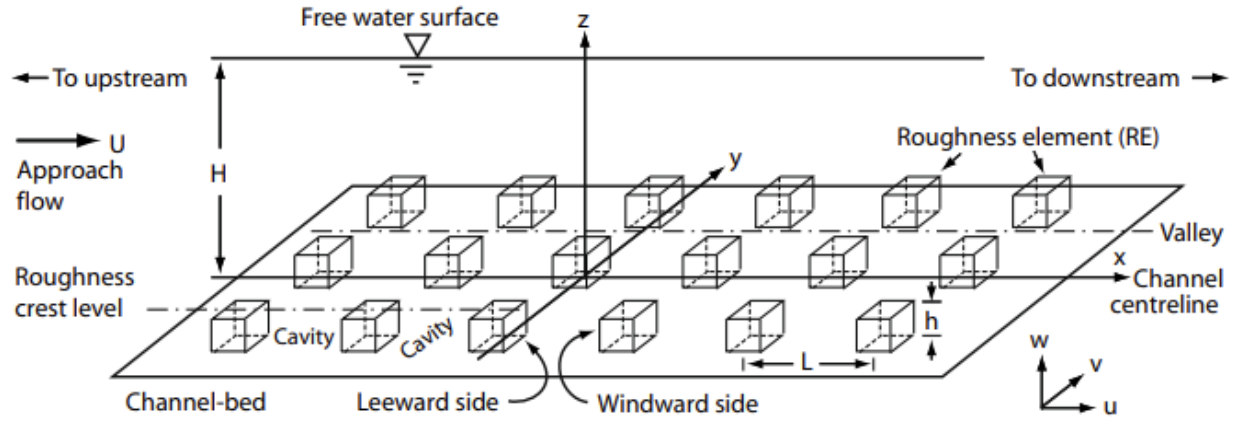


Figure 4.3.4-B: Roughness elements diagram showing the direction of flow, spacing of roughness elements (L), and roughness element height (h). Source: Li & Li (2020).

Baffles are an effective energy dissipation method for high-energy flows in culverts, chutes, or aprons with moderate slopes. These systems use a series of baffles, essentially raised roughness elements, installed along the floor to disrupt the flow, slowing it before and

accelerating it after each block, thereby reducing overall kinetic energy (Figure 4.3.4-A[b] and 4.3.4-C). When properly designed, this configuration can reduce flow velocities at the apron bottom to one-third of critical velocity. Baffled aprons are particularly useful for preventing excessive acceleration during vertical drops, rather than mitigating high approach velocities. Their performance remains effective with or without downstream tailwater, and they are generally resilient to trash or debris accumulation. Key design parameters include a culvert or chute slope between 25% and 50%, unit discharge under 60 cubic feet per second per foot, and approach velocity below critical velocity. Baffle height should be approximately 0.8 times the critical depth, with width and spacing around 1.5 times the height. Four (or more) rows of baffles are typically sufficient to establish flow control, and additional features such as recessed approaches or conventional riprap at the outlet may be incorporated to further enhance performance and prevent undermining (FHWA, 2006).

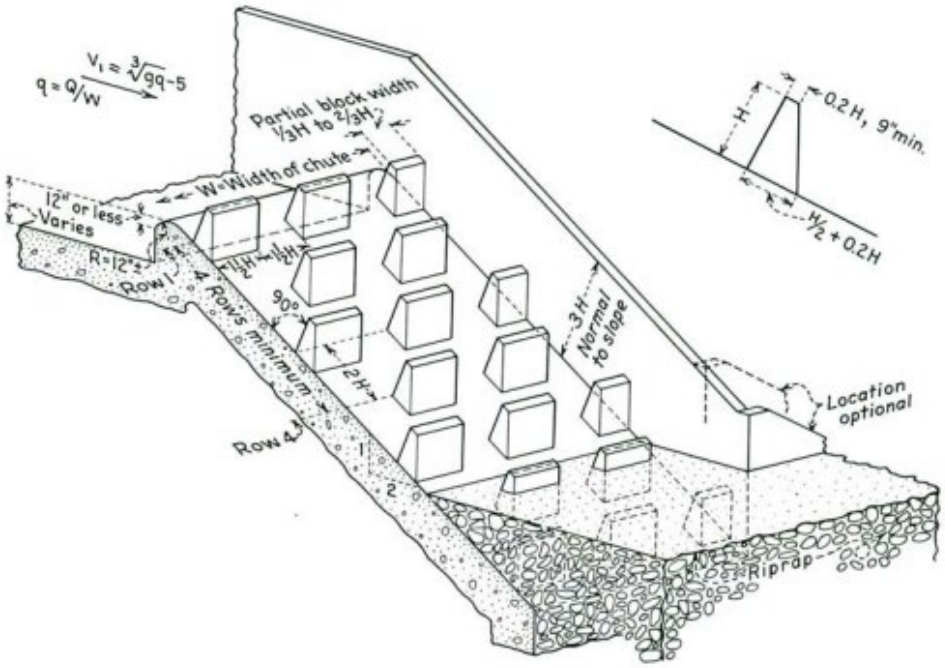


Figure 4.3.4-C: Baffled apron details. Source: Peterka (1978).

Broken-back culverts are used as energy dissipators by promoting the formation of a hydraulic jump within a flat runout section of culvert, thereby reducing the kinetic energy of high velocity flows. These culverts consist of a steep inlet section followed by a horizontal outlet section (Figure 4.3.4-D). A hydraulic jump forms when either the downstream momentum exceeds the upstream momentum, or when the supercritical flow in the steep section decelerates to a Froude number near 1.7 (Eqn 16, Section 4.3.2). The design goal is to ensure that the jump occurs and remains confined within the runout section, where it dissipates energy efficiently and minimizes downstream erosion. Engineers analyze flow

profiles to determine the location and length of the jump, ensuring it fits within the flat section. This method is particularly useful when external energy dissipation structures are not feasible. Broken-back culverts are best suited for mild slopes and require adequate runout length and tailwater conditions to complete the hydraulic jump internally. If conditions are not ideal, outlet modifications can be used to induce the jump (Thomason, 2019).

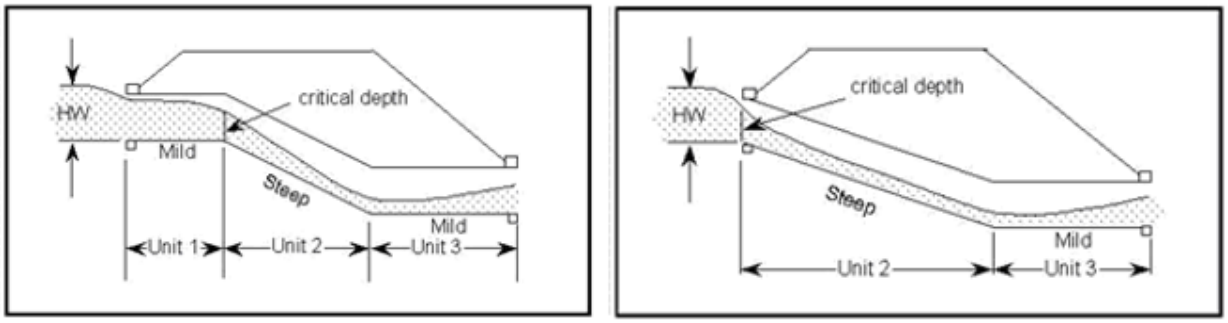


Figure 4.3.4-D: Three-unit broken back culvert diagram (left) and two-unit broken back culvert diagram (right). Source: Thomason (2019).

Weirs can be used as energy dissipators by inducing a hydraulic jump near the outlet of a culvert, effectively reducing the velocity of supercritical flows and minimizing downstream erosion. Typically installed in the horizontal runout section of a broken-back culvert about 10 feet upstream from the outlet, weirs span the full width of the culvert and are designed to trigger a hydraulic jump when the approach Froude number is between 2 and 7 (Eqn 16, Section 4.3.2). As water flows over the weir, it passes through critical depth and becomes supercritical, leading to a hydraulic jump that dissipates energy before the flow exits the culvert (Figure 4.3.4-A[c]). Drain holes in the weir prevent standing water upstream, and its placement allows for easier debris removal. Weirs are particularly effective when there is little or no tailwater and when used in rectangular culverts with weir heights between 0.7 and 4.2 times the upstream flow depth (FHWA, 2006). Their use reduces the need for extensive downstream channel protection.

Impact Basins are compact, engineered structures designed to dissipate the energy of high-velocity stormwater discharges, commonly installed at the outlets of culverts or spillways to prevent downstream erosion. Their primary mechanism involves directing the flow against a vertical hanging baffle, which redirects the water upward and backward, forming turbulent eddies as it interacts with the horizontal baffle section and basin floor; thus, significantly reducing runoff velocity. Unlike traditional hydraulic jumps, impact basins operate effectively without relying on tailwater conditions, although moderate tailwater levels can improve performance. Typical design features include a box-like concrete structure, a vertical baffle

with cleaning notches, a low-end sill (weir) to help maintain proper water levels, and riprap aprons (Section 4.3.6) downstream to manage residual velocity. Some installations also include wingwalls or a cutoff wall to prevent undermining and promote structural stability. A short support beneath the baffle enhances strength and contributes additional energy dissipation. Horizontal inlet sections are often recommended, even for culverts on moderate slopes, to improve flow orientation and ensure optimal performance. Although impact basins offer reliable energy dissipation across a range of flow conditions, they are not suitable in areas with high debris, and their use is typically limited to discharges under 400 cubic feet per second and velocities below 50 feet per second (FHWA, 2006; TDOT).



Drop structures are widely used for energy dissipation and erosion control in channels with steep gradients. Their primary function is to break a steep slope into a series of shorter, milder slopes and vertical drops, preventing the development of high velocity, erosive flows. Unlike methods that slow already fast-moving water, drop structures preemptively control channel slope to ensure high velocities never form. As water falls over the crest of each structure, the resulting kinetic energy is dissipated in a stilling basin (Section 4.3.7) or concrete apron designed to absorb impact, redirect flow, and induce turbulence. Drop structures are typically used in subcritical flow conditions, but considerations for upstream supercritical flow are also possible.

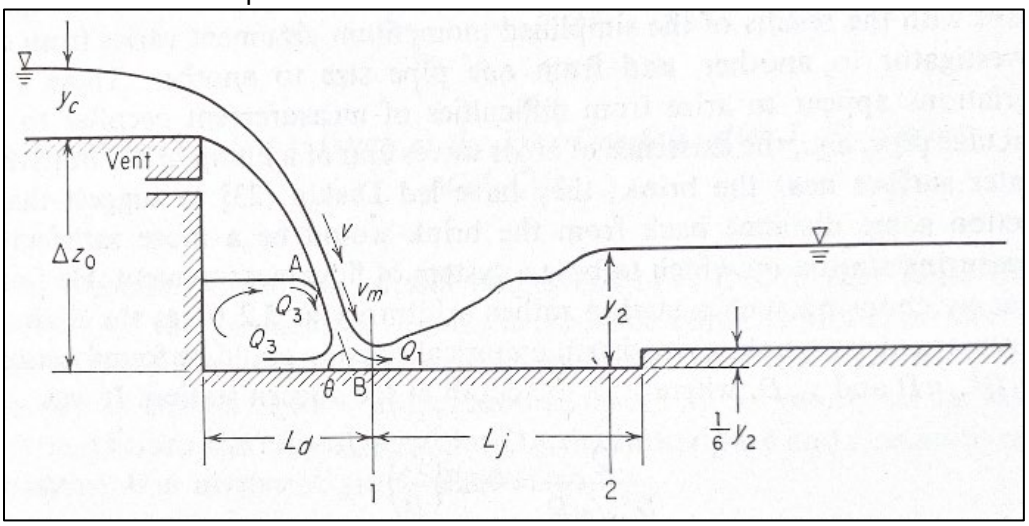


Figure 4.3.4-E: Geometries of a drop structure. Source: Henderson (1966).

Additional energy dissipators and resources for the design of the dissipators presented herein can be found in FHWA (2006), Admiraal & Zhang (2023), TDOT, and Thomason (2019).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here. For design examples, refer to *HEC-14* by FHWA (2006).

References

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4.3.5 Level Spreader



Source: Ohio State Stormwater Management Program

Definition and Purpose

A level spreader is a flow control measure that receives concentrated flow and converts it to sheet flow. It is essentially a broad, level outlet structure, such as a weir, that dissipates flow energy and downstream erosion potential. Additionally, a secondary benefit of level spreaders is that they can infiltrate runoff and reduce pollutants such as sediments and nutrients (Winston et al., 2011). Level spreaders are more effective when vegetation is incorporated into the design.

Appropriate Applications

Level spreaders are best suited for locations where sediment-free stormwater runoff can be discharged evenly onto stable, vegetated slopes without causing erosion. They are particularly effective where the downstream area has a uniform grade of 10% or less and is free of surface irregularities. Ideal sites include areas where the natural slope directs flow away from the project site and where the released runoff will not reconcentrate or be subjected to traffic. Level spreaders are also appropriate as outlets for temporary or permanent stormwater control features with concentrated flows. They are especially useful when it is necessary to avoid disturbing riparian buffers with an outfall or outlet protection.

Limitations and Maintenance

One of the primary limitations of level spreaders is the necessity for the weir lip to remain completely level; any irregularities in the weir profile can result in concentrated flows that undermine the device and contribute to erosion. This is particularly a concern during freeze-

thaw cycles. Level spreaders should never be installed at or above the finished grade of adjacent surfaces. Furthermore, level spreaders are not well-suited for distributing concentrated flows exceeding 30 cubic feet per second (NCDEQ, 2013; VDEQ, 2024).

During construction and until vegetation becomes established, inspections are most helpful after each storm to identify and repair any low points, cracks, or debris accumulation that could compromise performance. Regular tasks include maintaining the weir at zero grade, clearing debris, mowing to preserve healthy vegetation, and ensuring there is no subsidence or erosion downstream. In addition, removal of sediment buildup behind the level spreader should be done once the capacity has reached 50%. The spreader location needs to be accessible for easy maintenance yet separated from pedestrian and traffic routes.

Planning and Design Considerations

Implementing level spreaders on construction sites requires careful planning to ensure their effectiveness and longevity. These low-cost outlet structures work best in small drainage areas with concentrated flow rates under 30 cubic feet per second, making them well-suited for managing runoff from dikes, diversions, channels, swales, slope drains, etc. A completely level outlet lip is critical to maintain uniform sheet flow; any low points or irregularities may cause the flow to reconcentrate and trigger erosion. It is good practice to use a laser level for verification of proper installation. Standard practice allows for a tolerance of up to a quarter inch per 25 feet of length. In higher flow conditions, a rigid lip design helps sustain proper flow dispersion. Where runoff carries significant sediment, directing it first through sediment control measures helps prevent buildup and preserves the spreader function.

In addition to the level spread itself, the total system includes a channel and a vegetated filter strip (Section 4.4.3) as well as an optional forebay (Figure 4.3.5-A).

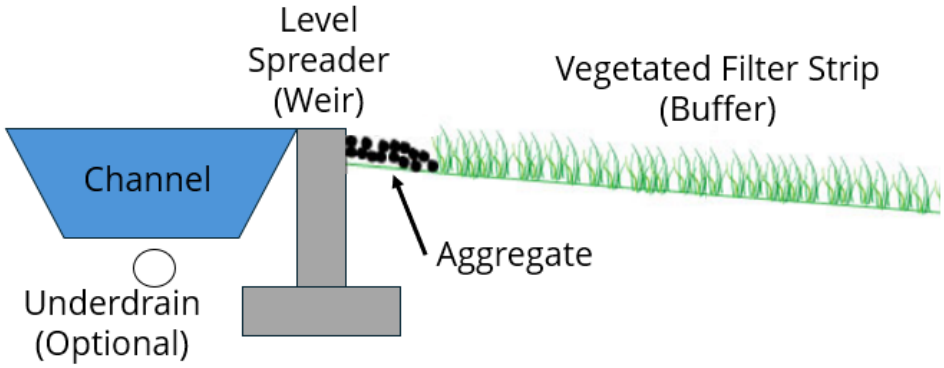


Figure 4.3.5-A: Level spreader schematic. Source: Ohio State Stormwater Management Program.

A forebay may be used as pretreatment before the level spreader. It is an excavated, bowl-shaped feature that slows the influent runoff and allows larger sediments and debris to settle and may be defined by a porous barrier such as a check dam, baffle, or rock berm. The forebay may be lined with conventional riprap to reduce erosion in the excavated area. Forebays dissipate energy and reduce the sediment accumulation behind the level spreader.

After stormwater passes through the forebay, it enters a concrete, rock, or grassed channel, the main body of the level spreader. The channel is a long, shallow impoundment that fills to the level of its lower side. The lower side (the *downslope side*) of the channel is constructed so that it is level along its full length. This lower side, or level spreader lip, is often constructed of concrete or rock so that it resists erosion. As stormwater enters the channel, it rises to smoothly meet the width of the level spreader to ensure uniform overflow. To ensure a smooth transition from the channel to the level spreader, a minimum channel width of six feet is often advised (VDEQ, 2024, TDOT), and depths can be specified from Table 4.3.5-A.

Table 4.3.5-A: Minimum channel depths (feet) for level spreaders based on design flows and downgradient slopes. Source: TDOT.

Design Discharge (cfs)	Downgradient Slope (%)			
	0 - 4	4 - 6	6 - 8	8 - 10
1	1.5	1.5	1.5	1.5
2	1.5	1.5	1.7	2
4	1.5	1.8	2.5	3
7	1.9	2.5	3.5	4.1
10	2.3	3.1	4.2	-
15	3	3.9	-	-
20	3.5	4.5	-	-
25	3.9	5.2	-	-
30	4.3	5.7	-	-

Finally, runoff enters a vegetated buffer. As runoff passes through the buffer vegetation, some of the water infiltrates. When properly vegetated, the buffer can remove sediment and nutrients from runoff before leaving the site and entering a downstream waterbody (Winston et al., 2011).

The level spreader structure is to be designed such that the channel behind it drains in approximately 24 hours. A general rule of thumb is to achieve a drawdown rate of half an inch per hour. The length of the level spreader, which acts as a broad crested weir, can be calculated using Eqn 17 (Section 4.3.3.1) by restricting a flow depth above the level spreader and calculating the peak flow rate using the design storm and approved methodologies

(Section 2.1.3). A flow depth of one foot has been recommended (Winston et al., 2011). Alternatively, and more simply, the weir length can be calculated by dividing the design peak flow rate by a unit weir flow rate (Table 4.3.5-B).

Table 4.3.5-B: Unit weir flow rates (cubic feet per second per linear foot) used to calculate level spreader weir lengths. Source: TDOT.

Downgradient Slope (%)	Unit Weir Flow Rate (cfs/LF)
0 - 4	0.49
4 - 6	0.2
6 - 8	0.07
8 - 10	0.04

Ideally, the ground surface on the downstream side of the level spreader should sit flush with the weir crest. Where this is not achievable, lay #57 stone or similar aggregate at a 4H:1V (or shallower) slope until the two are flush.

All level spreaders must discharge runoff in a straight or convex manner to decrease the chances of re-concentrated flow downgradient. (Figure 4.3.5-B). Furthermore, ensure the downgradient slope is promptly re-reseeded and mulched if the area was disturbed. Lastly, level spreaders can be constructed from various materials such as concrete, wood, and earth. It is usually ideal to line the level spreader with an erosion-resistant material if it is not constructed with concrete.

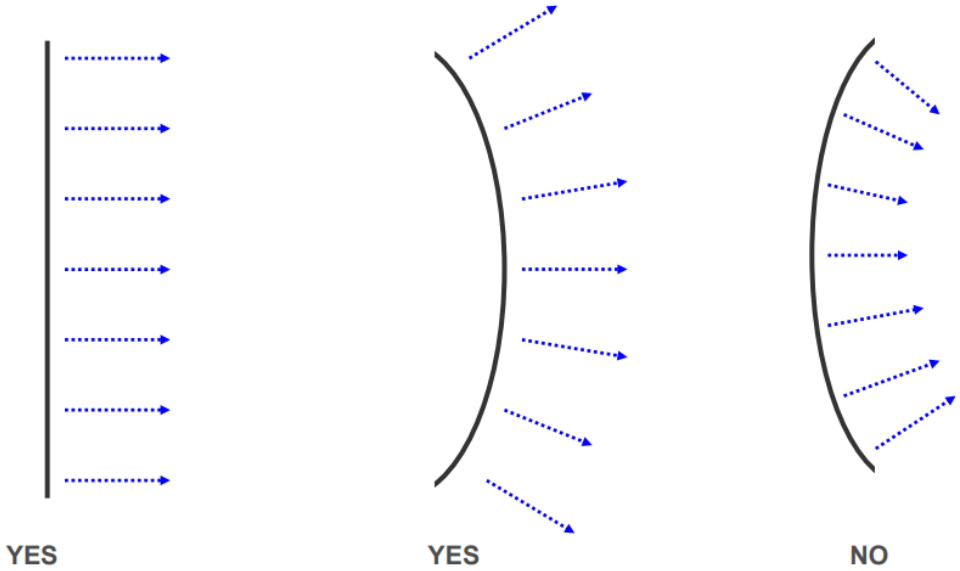


Figure 4.3.5-B: Allowable level spreader discharge patterns. Source: Ohio State Stormwater Management Program.

Example Application

-Example courtesy of TDOT-

Given:

A proposed area of 0.8 acres is to be developed, which drains to an ETW. A proposed ditch will convey a design peak flow of 4.8 cubic feet per second. A designer wishes to install a level spreader at the end of the ditch where the rest of the land is sloped at 1% towards the ETW.

Determine:

The depth of the channel upgradient of the level spreader and the length of the level spreader weir.

Solution:

Interpolating from Table 4.3.5-A yields a required depth of about 1.6 feet.

From Table 4.3.5-B, a unit weir flow rate of 0.49 cfs/LF is required for the design. By dividing the design flow rate of 4.8 cfs by the unit weir flow rate factor, a length of 9.8 feet is computed. Round up to 10 feet of length.

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4.3.6 Riprap Apron





Source: TDEC

Definition and Purpose

Riprap aprons are a form of outlet protection placed downstream of drainage infrastructure, typically culverts or channels. They are designed to prevent erosion at the drainage outlet by reducing flow velocity and dissipating energy.

Appropriate Applications

These measures are generally applicable at the outlet of most any storm drain, culvert, paved channel, or slope drain where outlet velocity and shear stress exceed the capacities of downgradient natural materials. Though riprap aprons are typically permanent measures, they can also be applied at the outlet of temporary structures, such as a temporary diversion.

Limitations and Maintenance

Riprap aprons are not an advisable measure when the velocity at the respective outlet exceeds 15 feet per second. Additionally, apron lengths can become excessively long when highly erosive flows are anticipated, which may lead to infringements on property lines (TDOT). Riprap apron lengths greater than 50 feet are less desirable because such structures would add complexity in obtaining environmental permits. In such cases, a stilling basin

(Section 4.3.7) may be more ideal. Conversely, if aprons are too short, they become ineffective and increase the potential for erosion downstream (FHWA, 2006).

Inspect riprap outlet structures after heavy rains to see if any erosion around or below the riprap has taken place or if the stones have been dislodged. Immediately repair all damages. Remove and replace the conventional riprap if the apron is fully clogged with sediment or if sediment is at risk of leaving the site.

Planning and Design Considerations

Outlets, pipes, channels, etc., typically discharge water at velocities capable of causing scour, channel degradation, or embankment failure. A riprap-lined apron functions as a transition structure that dissipates the flow energy, reduces velocity to non-erosive levels, and prevents undermining of the outlet. The apron lining typically consists of loose conventional riprap, or grouted riprap or concrete, depending on site conditions and flow intensity. In locations where very high flows would demand excessive apron lengths, alternatives such as plunge pools or stilling basins offer more compact energy dissipation. Effective planning and implementation of such structures requires constructing the apron at zero grade and extending it downstream until stable hydraulic conditions are reached.

Rock gradation as well as apron dimensions are calculated based on anticipated flow rates from the design storm. If the calculated gradation of stone is not available, round up to the next available D_{50}. Because riprap aprons are typically permanent measures, they should be designed to handle the same design flow rate as the drainage structure they serve. When they are designed as a temporary structure, temporary aprons can be designed using the 2-year, 24-hour or 5-year, 24-hour peak flow rate, depending on the status of the receiving water. Rock gradation (i.e., D_{50}) can be computed utilizing Eqn 22 when using conventional riprap for an apron draining a circular culvert:

$$D_{50} = 0.2 \times D \times \left(\frac{Q}{\sqrt{g} \times D^{2.5}}\right)^{4/3} \times \frac{D}{TW} \quad \text{(Eqn 22)}$$

where D_{50} is the stone size in which 50% is finer by weight (feet), Q is the design discharge (cubic feet per second), D is the circular culvert diameter (feet), TW is the tailwater depth (feet), and g is acceleration due to gravity (32.2 feet per square second). Use Appendix B of FHWA (2006) if TW computations are required; however, TW should be limited to between 0.4 ×D and 1.0 × D, using 0.4 × D if TW is unknown (FHWA, 2006). When flow is supercritical [i.e., the Froude Number (Eqn 16, Section 4.3.2) is greater than one), the culvert diameter is adjusted following Eqn 23:

$$D' = \frac{D + y_n}{2} \quad \text{(Eqn 23)}$$

where D' is the adjusted culvert rise (feet) and y_n is the normal (supercritical) depth in the culvert (feet). Apron dimensions (length and depth) can be obtained from Table 4.3.6-A once the D_{50} has been calculated.

Table 4.3.6-A: Riprap apron length and depth dimensions based on D_{50}. Source: FHWA (2006).

D_{50} (in)	Apron Length[a] (feet)	Apron Depth (feet)
5	4 × D	3.5 × D_{50}
6	4 × D	3.3 × D_{50}
10	5 × D	2.4 × D_{50}
14	6 × D	2.2 × D_{50}
20	7 × D	2.0 × D_{50}
22	8 × D	2.0 × D_{50}

[a]D is the culvert rise.

The width of the apron (at the most upstream end of the apron) should be three times the diameter of the culvert. If runoff is not channeled such that it can only travel in the direction of the length of the apron, the apron width should linearly increase with length (i.e., trapezoidal in shape). The width on each side of the apron should increase by one foot per three feet of apron length. Apron width specifications are shown in Figure 4.3.6-A.

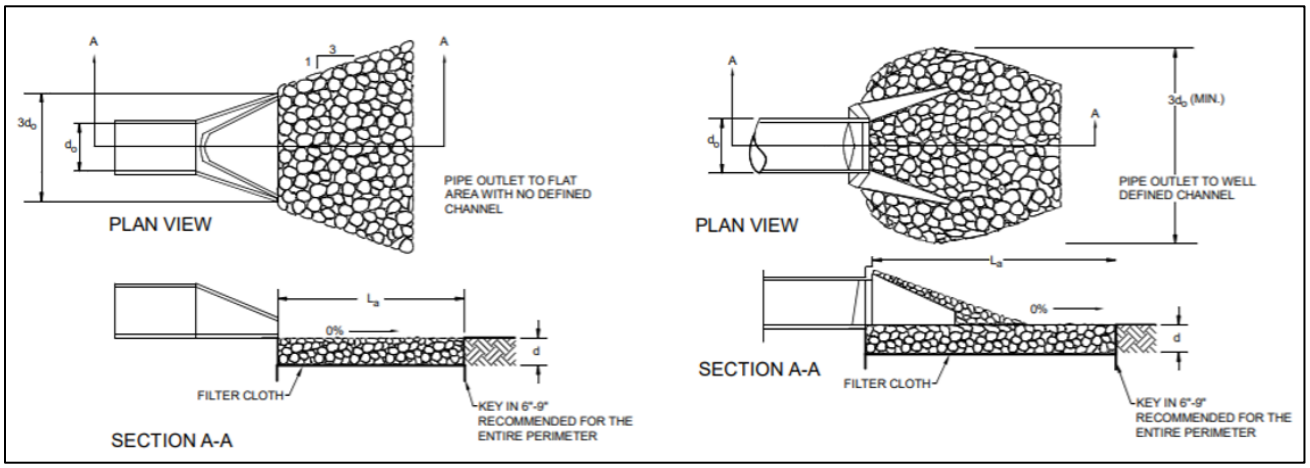


Figure 4.3.6-A. Apron width details. Adapted from VDEQ (2024).

In cases where TW > 1.0 × D and Eqn 22 is not appropriate, consider implementing a stilling basin (Section 4.3.7) or an energy dissipator (Section 4.3.4) upstream of the apron. In the latter case, use Eqn 24 to size the conventional riprap:

$$D_{50} = \frac{0.692}{S-1} \times \frac{V^2}{2 \times g} \qquad \text{(Eqn 24)}$$

where S is the specific gravity of the riprap (typically 2.65), and V is the velocity at which water leaves the dissipator (feet per second). Figure 4.3.6-B should be used to determine velocity downstream of the culvert, which is then used to determine the apron length.

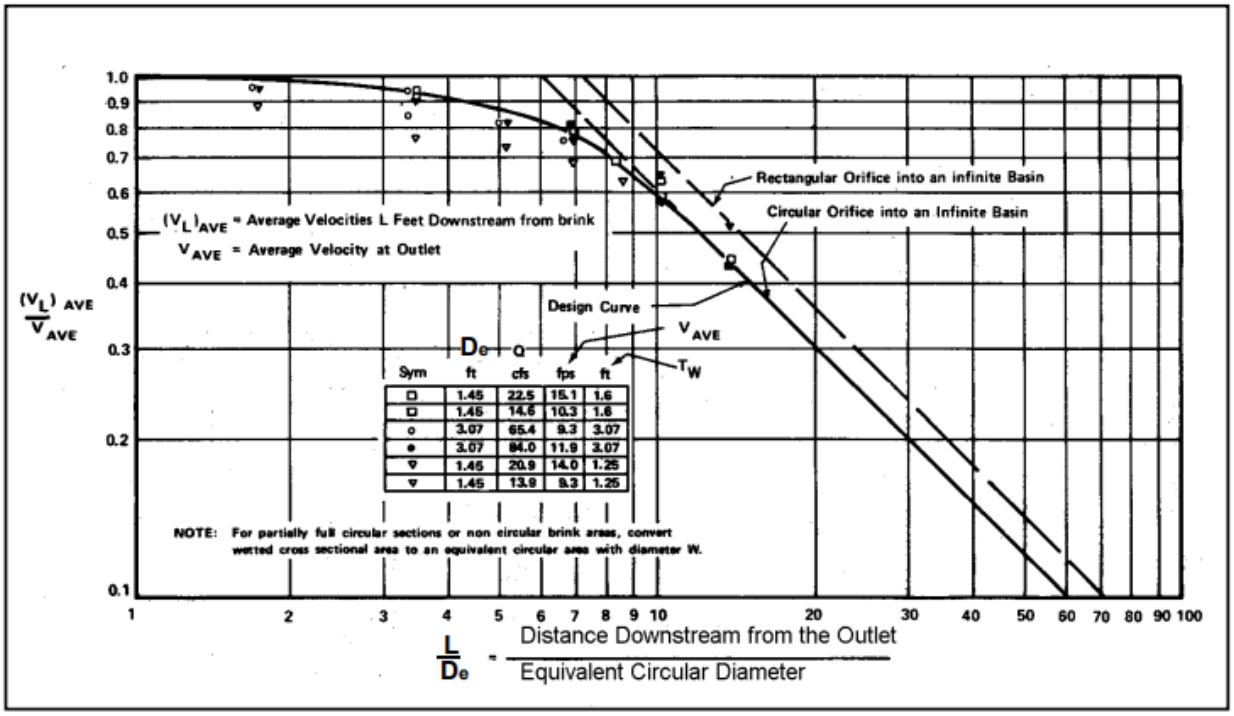


Figure 4.3.6-A: Distribution of centerline velocity for flow from submerged outlets. In this case, it can be used to determine the downstream velocity flow. Source: FHWA (2006).

Non-woven geotextile fabric or stone filter layers should be laid between the soil surface and conventional riprap to prevent piping, settling, and sediment migration as expressed in Section 4.2.6.4 (Hard Armoring). If using a geotextile, it is recommended to meet the requirements found in AASHTO M288 for a Class 2 separation geotextile and lay with such that no voids between the soil surface and fabric are present (ALSWCC, 2018).

Example Application

-Example from FHWA (2006)-

Given:

A design flow rate of 85 ft^3/s, culvert diameter of 5 ft, and tailwater depth of 1.6 ft.

Determine:

The D_{50} and dimensions for a riprap apron.

Solution:

Step 1 – Determine the stone gradation: First, verify that the tailwater is within range. TW/D = 1.6/5 = 0.32. This is less than 0.4 × D, therefore, use TW = 0.4 × D = 0.4 × 5 = 2 ft. Apply Eqn 22.

$$D_{50} = 0.2 \times D \times \left(\frac{Q}{\sqrt{g} \times D^{2.5}}\right)^{4/3} \times \frac{D}{TW}$$

$$D_{50} = 0.2 \times 5 \times \left(\frac{85}{\sqrt{32.2} \times 5^{2.5}}\right)^{4/3} \times \frac{5}{2}$$

$$D_{50} = 5.2 \text{ inches.}$$

Step 2 – Determine riprap class: This may vary from supplier to supplier. Select the class of riprap that has a D_{50} at least as large as what was calculated in Step 1. For the purposes of this example, assume a supplier has a class of conventional riprap with a D_{50} of 6 in.

Step 3 – Determine apron dimensions: Using Table 4.3.6-A, determine the length, depth, and width (at apron end) of the apron.

$$\text{Length} = 4 \times D = 4 \times 5 = 20 \text{ ft}$$

$$\text{Depth} = 3.3 \times D_{50} = 3.3 \times 6 = 19.8 \text{ in} = 1.65 \text{ ft}$$

$$\text{Width} = 3 \times D + (2/3) \times \text{Length} = 3 \times 5 + (2/3) \times 20 = 28.3 \text{ ft.}$$

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

FHWA. (2006). *HEC 14: Hydraulic Design of Energy Dissipators for Culverts and Channels.*

TDOT. *Drainage Manual Ch10*.

VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.3.7 Stilling Basin



Source: TDEC

Definition and Purpose

Stilling basins are constructed riprap (typically conventional riprap) basins composed of a flat-bottom pool and an apron (Section 4.3.6). These permanent structures are located at drainage structure outfalls, where concentrated flows plunge into one end of the pool, forming a hydraulic jump at the other end against the apron. As a result, flow will generally be well dispersed as it leaves the basin, thereby preventing scour at the structure outlet and minimizing downstream erosion.

Appropriate Applications

This measure may be applied at drainage structure outlets where the velocity at the design discharge is 15 feet per second or greater. It may also be used at structures with lower outlet flow velocities where the required length of a riprap apron would be excessive or infeasible due to site conditions (TDOT).

Limitations and Maintenance

The use of this measure is not specifically limited; however, consider the available right-of-way area, the availability of materials, and the difficulties that could be encountered when implementing this measure on a project.

Additional permits that may be required include an ARAP, a Section 404 permit from USACE, a TVA 26a permit, or a local permit. If any additional permit is required, its conditions must be followed in conjunction with the CGP.

Planning and Design Considerations

In contrast to temporary EPSC measures, which are usually designed using lower peak flow rates (i.e., 2 year-24 hour or 5 year-24 hour), a stilling basin is considered a permanent structure and should be designed using the same flow rate used to design the drainage structure it is intended to serve (TDOT). The basin dimensions may be modified as required to match the configuration of the drainage structure end treatment.

The overall design process should match that of FHWA (2006), and designers are referred to that source for explicit design details. Eqn 25 may be used to compute the required depth of a riprap basin:

$$\frac{H1}{d_e} = 0.86 \times \left(\frac{D_{50}}{d_e}\right)^{-0.55} \times \left(\frac{V_o}{\sqrt{g \times d_e}}\right) - C_o \qquad \text{(Eqn 25)}$$

where H1 is the dissipator pool depth (feet), d_e is the equivalent brink (outlet) depth (feet), and D_{50} is the median rock size by weight (feet), such that H1 divided by D_{50} is greater than or equal to two (Figure 4.3.7-A). Because of this H1:D_{50} ratio condition, designing riprap basin depth is an iterative approach. If the ideal D_{50} gradation is not available, round up to the next available size. Further, this equation contains a correction factor, C_o, which varies with tailwater depth (TW). Use Appendix B of FHWA (2006) if TW computations are required. C_o is computed by one of two sets of equations (Eqn 26 or Eqn 27) depending on whether a more conservative design is desired. In general, Eqn 26 will result in a basin depth one to two feet greater than Eqn 27; however, it will also allow a basin to be implemented in a greater number of scenarios and therefore may be more ideal for use.

$$\begin{array}{cc} C_o = 1.4 & TW:d_e < 0.75 \\ C_o = 4 \times (TW/d_e) - 1.6 & 0.75 < TW:d_e < 1 \\ C_o = 2.4 & TW:d_e > 1 \end{array} \qquad \text{(Eqn 26)}$$

$$\begin{array}{cc} C_o = 2 & TW:d_e < 0.75 \\ C_o = 4 \times (TW/d_e) - 1 & 0.75 < TW:d_e < 1 \\ C_o = 3 & TW:d_e > 1 \end{array} \qquad \text{(Eqn 27)}$$

Theoretically, the most efficient design would be to find the rock gradation such that the value of the H1:D_{50} ratio is a close to two as possible. For the majority of scenarios where a riprap stilling basin will be feasible, the value of H1 determined by Eqn 25 will be between one and a half and two and a half feet, regardless of the selected C_o (i.e., class A-1 conventional riprap will most commonly be used for stilling basins). Conventional riprap class B or C will be used to construct stilling basins only for comparatively large culvert installations. As a reminder, D_{50} size for class A-1, B, and C conventional riprap is approximately nine, 15, and 20 inches, respectively. For many smaller culverts, the value of H1 determined by Eqn 25 will be zero or negative, especially where the TW is more than approximately 85% of the equivalent depth at the culvert outfall. It is recommended that a

riprap apron be considered for these situations (TDOT). Once the basin depth has been established, all other basin dimensions can be inferred from Figure 4.3.7-A.

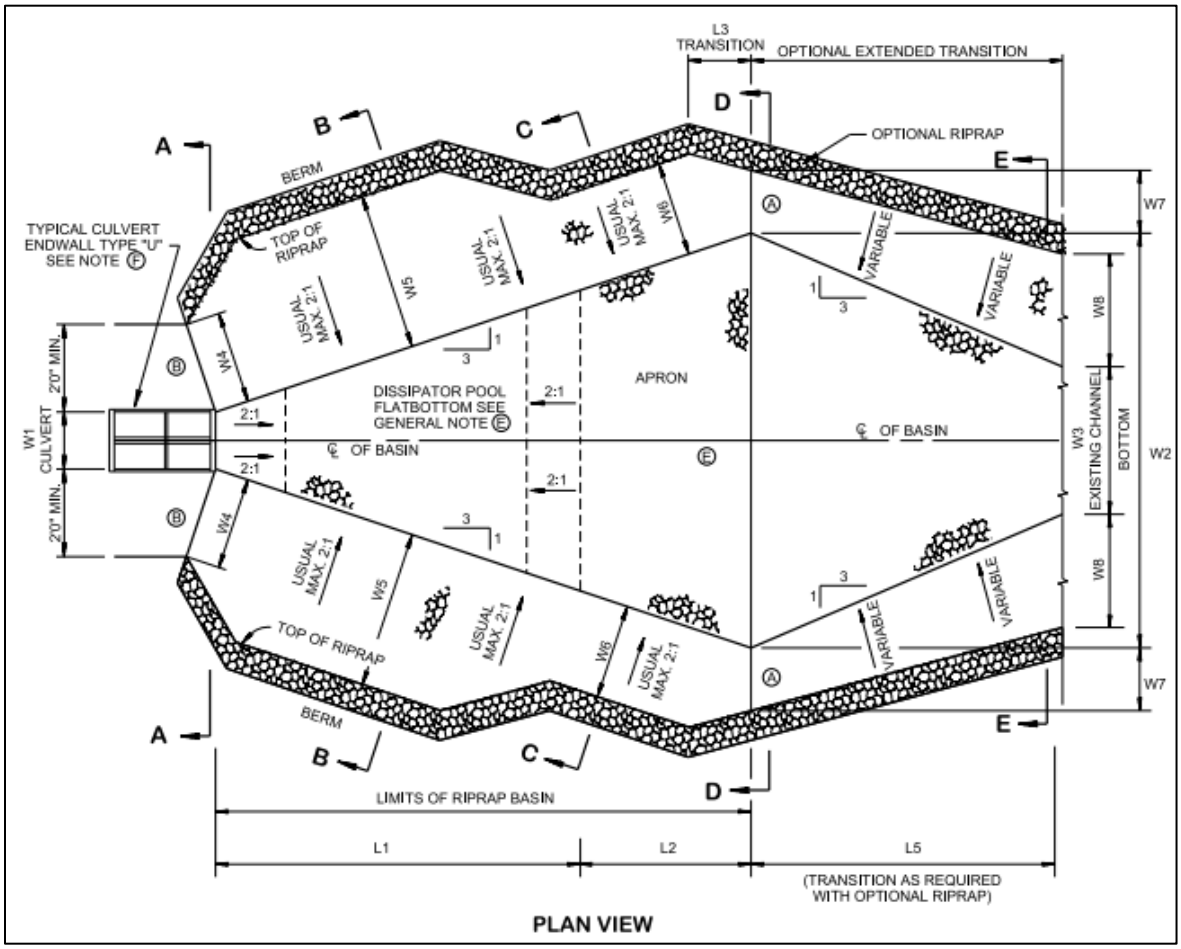


Figure 4.3.7-A: Stilling basin details. Source: TDOT.

This measure may be used as an alternative to a riprap apron where the required length of the apron would require additional right-of-way or permanent drainage easements. Furthermore, geotextile fabric should be placed beneath the conventional riprap used to construct a basin and meet all required standard specifications (TDOT).

Example Application

-Example courtesy of TDOT-

Given:

A culvert has been designed with the following characteristics:

- Design discharge of 100 cubic feet per second

- Diameter of 60 inches
- Inlet elevation at 601.5 feet
- Outlet elevation at 600 feet
- Length of 100 feet
- Endwall is Type "U"

The channel downstream of the culvert is trapezoidal, having a bottom width of four feet, a depth of one and a half feet, 6H:1V side slopes, and a slope of 0.015 feet per foot. The Manning's n value of the channel is 0.035. The stream carries a heavy load of sediments, small branches, and corn stalks.

Determine:

Whether the site is suitable for a stilling basin based on the culvert outlet hydraulics, tailwater depth, and debris load. If so, determine the proper class of conventional riprap and stilling basin dimensions.

Solution:

This site is a candidate for a riprap stilling basin because of the heavy debris load. Another type of dissipator might become clogged, thus reducing its effectiveness. Furthermore, it is judged that allowing a natural scour hole to form at this site could possibly have undesirable results. Other factors affecting whether a riprap basin would be suitable for this site will be investigated as the design progresses. As much as possible, the steps below follow the procedure provided in FHWA (2006).

Since the outlet velocity, Vo, is greater than 5 ft/sec, some form of erosion protection should be provided at the culvert outlet. In order to assess the suitability of a riprap stilling basin for the site, the following information is collected from the hydraulic analysis of the culvert:

- Brink depth, d_o, of 2.05 feet
- Outlet velocity, V_o, of 13.17 feet per second
- Tailwater depth, TW, of 1.55 feet
- Tailwater velocity, V_n, of 4.85 feet per second.

Step 1 – Find the TW:D_e ratio: The equivalent depth, d_e, at the culvert outfall is computed from Eqn 9-2 in TDOT:

$$d_e = (\frac{A_o}{2})^{0.5} = (\frac{7.68}{2})^{0.5} = 1.96 \text{ ft}$$

Step 2 – Find the appropriate C_o value: The C_o value is found using Eqn 26 or Eqn 27 based on the ratio of TW:d_e, which is 0.79.

The guidance herein indicates that a riprap apron should be used in place of a basin where the tailwater depth is greater than 75% of the brink depth. However, given the size of the pipe, high outflow velocity and a need to minimize the length of the riprap structure, it is decided to continue with the riprap basin design. If the computed basin depth, H1, is not at least twice the D_{50} of the selected conventional riprap, a riprap apron will be specified for the site.

As mentioned, Eqn 26 is often more ideal and therefore implemented herein.

$$C_o = 4 \times (\frac{TW}{d_e}) - 1.6 = 4 \times \frac{1.55}{1.96} - 1.6 = 1.56$$

Step 3 – Determine proper stone size and basin depth: Because class A-1 conventional riprap (D_{50} of 0.75 feet), is the most common class of riprap for a stilling basin, this is the first iterative input for Eqn 25:

$$\frac{h_1}{d_e} = 0.86 \times (\frac{D_{50}}{d_e})^{-0.55} \times (\frac{V_o}{\sqrt{g \times d_e}}) - C_o$$

$$\frac{h_1}{1.96} = 0.86 \times (\frac{0.75}{1.96})^{-0.55} \times (\frac{13.17}{\sqrt{32.2 \times 1.96}}) - 1.56$$

$$H1 = 1.62 \text{ feet.}$$

Now that H1 is solved, ensure the ratio of $H1:D_{50}$ is greater than or equal to two.

$$H1:D_{50} = 1.62/0.75 = 2.16$$

Since this result is slightly greater than two, it is determined that a riprap stilling basin with class A-1 conventional riprap will provide an efficient design.

Step 4 – Determine all other basin dimensions: Once the basin depth has been determined, all other basin dimensions can be inferred from the standard drawing.

W1 refers to the width of the basin floor at the culvert outfall. This site will be provided with a Type "U" endwall, thus: W1= 5.0 feet.

L1 refers to the length of the pool portion of the basin. FHWA (2006) recommends this length be the greater of 10 times H1 or 3 times W1. Thus:

$$10\times H1 = 10\times 1.62 = 16.2 \text{ feet}$$

or

$$3\times W1 = 3\times 5 = 15.0 \text{ feet.}$$

So: L1 =17.0 feet (rounded).

L2 refers to the length of the apron portion of the basin. FHWA (2006) recommends this length be the greater of 5 times H1 or W1. Thus:

$$5\times H1 = 5\times 1.62 = 8.1 \text{ feet}$$

or

W1= 5 feet.

So: L2 = 9.0 feet (rounded).

The standard drawings require the elevation of the basin floor, including both the pool and the apron, to be below the natural stream elevation at the end of the apron. The fall in the stream over the length of the basin is computed as:

$$\text{Fall} = (L1 + L2) = (17.0 + 9.0) \times 0.015 = 0.39 \text{ feet}$$

Occasionally, the designer may be presented with a situation in which the computed fall is very close to or even greater than H1. In that situation, it would be necessary to redesign the culvert with a lower outlet elevation and begin the riprap basin design again from Step 1. However, in this situation, since the computed fall is sufficiently less than H1, the design may proceed “as-is.” Since the slopes at the upstream and downstream ends of the pool are 2:1, the length from the culvert outfall to the basin floor is 2×1.62 = 3.24 feet, and the length of the transition from the basin floor to the apron is (1.62 −0.39)×2 = 2.46 feet. Thus, the length of the pool bottom is 17.0 − 3.24 − 2.46 =11.3 feet.

W2 refers to the width of the pool at the end of the apron. Since the floor expands at a 3:1 ratio on both sides, W2 is computed as:

$$W2 = W1 + (L1 + L2) \times (2/3) = 5 + (17 + 9) \times (2/3) = 22 \text{ feet}$$

H2 refers to the height at the top of the basin wall above the elevation of the culvert outfall. FHWA (2006) recommends the top of the basin should provide at least 1 foot of freeboard above the brink depth, do. Thus:

$$H2 = d_o + 1.0 = 2.05 + 1.0 = 3.1 \text{ feet (rounded)}$$

W4 refers to the width of the basin side slopes at the culvert outfall. Since the top of the basin is 3.1 feet above the outfall and the side slopes of the basin are 2:1,

$$W4 = 2 \times 3.1 = 6.2 \text{ feet}$$

W5 refers to the width of the basin side slopes above the floor of the pool. The top of the basin 4.8 feet above the floor of the pool, since the side slopes are also 2H:1V at this point:

$$W5 = 2 \times 4.8 = 9.6 \text{ feet}$$

W6 refers to the width of the basin side slopes above the floor of the apron. The distance to the top of the basin is equal to H2 plus the fall of the stream previously computed. Since the side slopes are still 2H:1V at this point:

$$W6 = (H2 + \text{Fall}) \times 2 = (3.1 + 0.39) \times 2 = 7 \text{ feet}$$

D1 refers to the thickness of the conventional riprap layer beneath the pool portion of the basin. The standard drawings indicate that the conventional riprap layer beneath the pool should be somewhat thicker than the conventional riprap layer beneath the apron. This varies from the recommendation in FHWA (2006), which indicates the riprap layer needs to be thicker only beneath the transition from the culvert outfall to the floor of the pool. Thus, the thickness criteria provided FHWA (2006) will be applied to the entire pool of the basin. Based on this approach, D1 will be the maximum of 3 times the median stone size, D_{50}, 2 times the maximum stone size, D_{max}, or the minimum thickness of the conventional riprap layer as specified in the Standard Specifications of 18 inches.

$$3 \times D_{50} = 3 \times 0.75 = 2.25 \text{ feet}$$

or

$$2 \times D_{max} = 2 \times 1.25 = 2.5 \text{ feet}$$

Thus, D1 = 2.5 feet.

D2 refers to the thickness of the conventional riprap layer beneath the apron portion of the basin. FHWA (2006) recommends that this layer be equal to the maximum of 2 times the median stone size, D_{50}, 1.5 times the maximum stone size, D_{max}, or the minimum thickness of the conventional riprap layer as specified in the Standard Specifications, as 18 inches.

$$2 \times D_{50} = 2 \times 0.75 = 1.50 \text{ feet}$$

or

$$1.5 \times D_{max} = 1.5 \times 1.25 = 1.9 \text{ feet}$$

This result may be rounded so that D2 = 2.0 feet

D3 and L4 refer respectively to the depth and length of a conventional riprap key, which is provided at the downstream end of the apron. The standard cut-off wall depth for a concrete structure is 3 feet. Furthermore, D3 and L4 should be approximately equal. Thus, the values of D3 and L4 will both be 3 feet.

Step 5 – Determine the transition dimensions: The width of the basin at the downstream end of the apron, W2, is 22 feet, which is greater than the natural channel bottom width of 4 feet. Thus, a transition will be provided to allow the basin cross-section at the downstream end of the apron to be warped to match the existing channel configuration. Thus, the following dimensions are determined for the transition.

W3 refers to the width of the transition at the downstream end. This should be equal to the existing channel width of 4 feet, therefore,

$$W3 = 4 \text{ feet.}$$

L5 refers to the length of the transition from the downstream end of the apron to the basin outlet. The TDOT standard drawings indicate that the transition should be at a rate of 3:1, so L5 is computed as:

$$L5 = (W2 - W3) \times (3/2) = (22 - 4) \times (3/2) = 27 \text{ feet.}$$

W7 refers to the width of the basin side slopes above the floor of the transition just downstream of the end of the apron. At this point, the top of the basin has already been transitioned from H2 computed in Step 5 to the height of the natural channel banks above the channel bed. This transition takes place over a distance, L3, which will be discussed below. At this site, the natural channel is 1.5 feet deep. Since the side slopes of the basin are still 2H:1V at this location,

$$W7 = 2 \times 1.5 = 3.0 \text{ feet.}$$

W8 refers to the width of the basin side slopes at the outlet from the transition. The side slopes of the basin are continuously warped from 2H:1V at the beginning of the transition to the natural channel side slopes at the end of the transition. Since the natural channel side slopes are 6H:1V and the channel depth is 1.5 feet,

$$W8 = 6 \times 1.5 = 9.0 \text{ feet.}$$

L3 refers to the length over which the top of the riprap basin transitions from H2 to the height of the natural stream bank. This transition occurs just upstream from the end of the apron and begins at the point where the top of the slope through the transition intersects the top of the basin above the apron. This length is computed from:

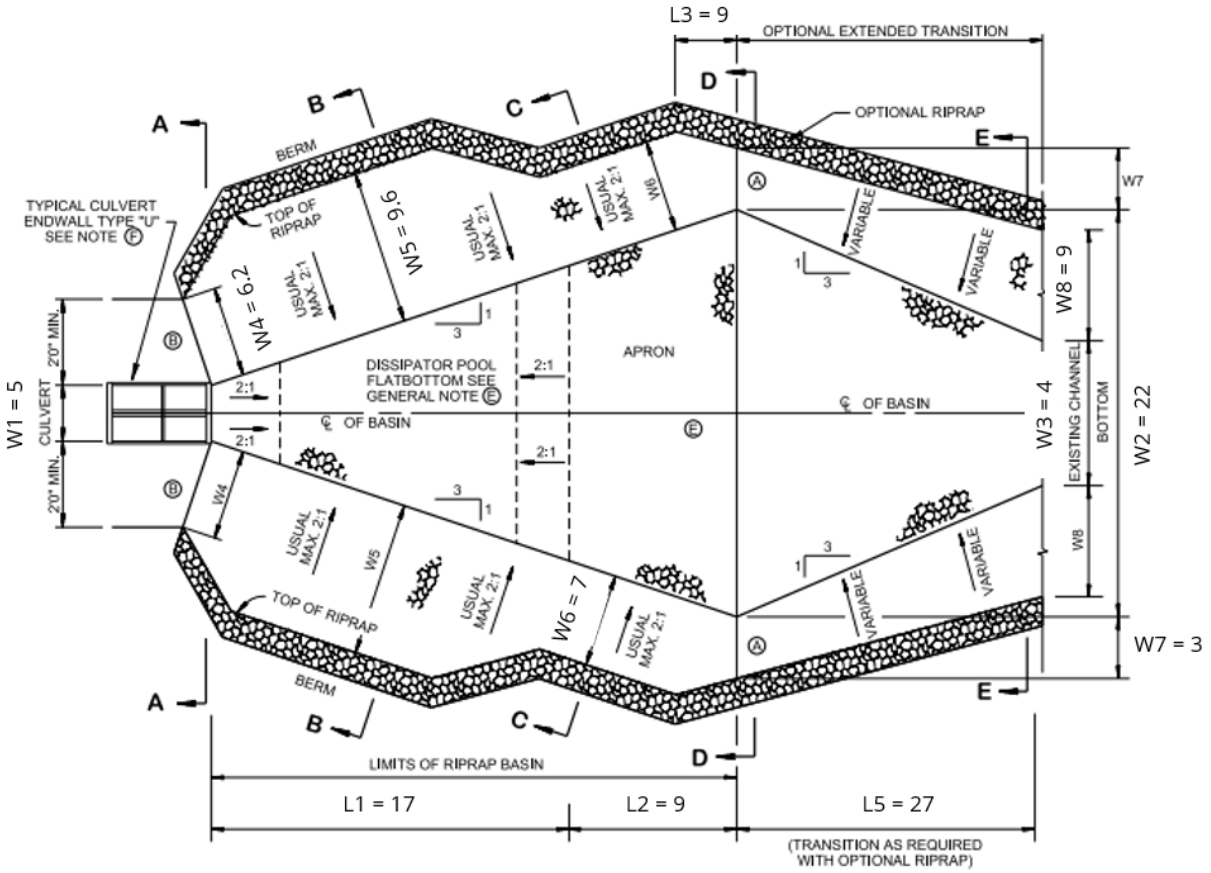
$$L3 = \frac{\left(\frac{W2}{2} + W6\right) - \left(\frac{W2}{2} + W7\right)}{\frac{1}{3} + \frac{\left(\frac{W2}{2} + W7\right) - \left(\frac{W3}{2} + W8\right)}{L5}} = \frac{\left(\frac{22}{2} + 7\right) - \left(\frac{22}{2} + 3\right)}{\frac{1}{3} + \frac{\left(\frac{22}{2} + 3\right) - \left(\frac{4}{2} + 9\right)}{27}} = 9 \text{ feet}$$

As described above, the height of the top of the basin above the apron is 3.1 feet, and the height of the channel bank is 1.5 feet. Thus, a transition of 1.6 feet will occur over the distance L3. This represents a slope of about 4.5H:1V. This slope will be adequate.

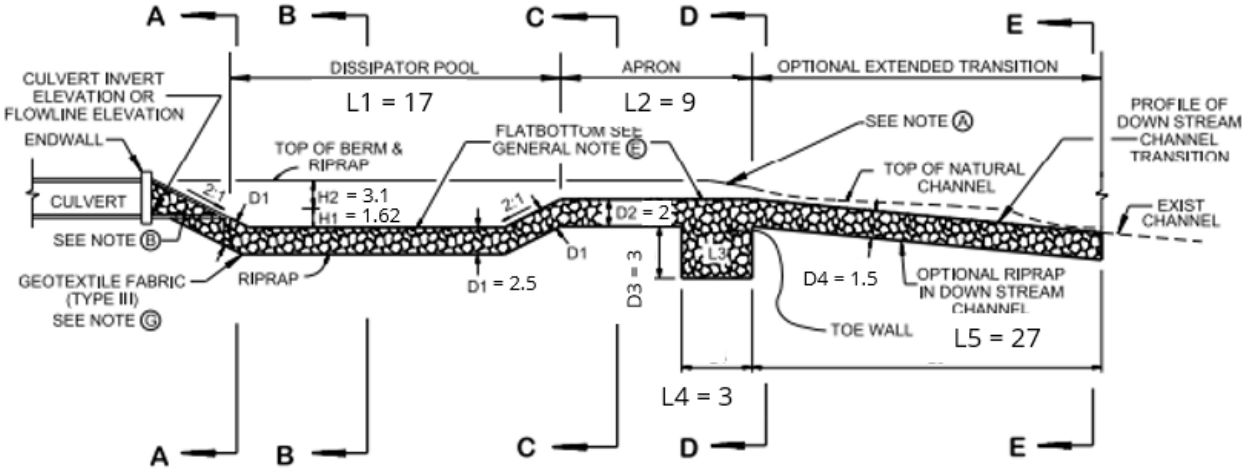
D4 refers to the thickness of the conventional riprap layer beneath the transition. This should be equal to the minimum layer thickness recommended for Class A1conventional riprap, or 1.5 feet.

The following table and drawings summarize the results (in feet, unless otherwise specified) for this design example.

Dimension	Value
Station	1 + 90
Distance	62
Direction	Lt.
Culvert Size (in)	60
Culvert Length	100
W1	5
W2	22
W3	4
W4	6.2
W5	9.6
W6	7
W7	3
W8	9
H1	1.62
H2	3.1
L1	17
L2	9
L3	9
L4	3
L5	27
D1	2.5
D2	2
D3	3
D4	1.5



PLAN VIEW



ELEVATION VIEW

References

FHWA. (2006). *HEC 14: Hydraulic Design of Energy Dissipators for Culverts and Channels.*
TDOT. *Drainage Manual Ch9*.
TDOT. *Drainage Manual Ch10*.

4.3.8 Surface Roughening



Source: TNWRRC

Definition and Purpose

Surface roughening is a temporary measure that involves mechanically creating horizontal depressions, furrows, or irregularities up and down a slope. Using methods such as stair-stepping, contour tracking, grooving, etc., with heavy machinery, surface roughening disrupts the flow of runoff traversing down a slope, thereby reducing its velocity and erosion potential. Furthermore, these horizontal roughenings can entrap sediment and improve the microenvironment by enhancing moisture retention, moderating temperature fluctuations, and providing mechanical protection for seed, lime, and fertilizer applications. Surface roughening is often performed in conjunction with grade controls (Section 4.2.4).

Appropriate Applications

Broadly, surface roughening is applicable to most any slope. Ideally, surface roughening can be used in conjunction with stabilization practices such as hydroseeding (Section 4.2.6.5), mulching (Section 4.2.6.9), and seeding (Sections 4.2.6.11 and 4.2.6.12) (KTC, 2015; USEPA, 2021). However, surface roughening is not intended to be used in conjunction with rolled erosion control products (Section 4.2.6.6).

Limitations and Maintenance

Avoid surface roughening on rocky, densely compacted, or clayey soils, where equipment may not be able to create adequate surface depressions, or where the soil composition

prohibits sufficient infiltration. The (over)use of tracked equipment can lead to excessive soil compaction, which may hinder seed germination and infiltration capacities (USEPA, 2021). Always ensure the equipment used to roughen surfaces traverses up and down the slope. Traversing across the slope creates depressions that run along the slope, thereby creating small rills or pathways for water to freely travel downslope and gain velocity (example shown to the right). Surface roughening is also limited to smaller precipitation events, as large events may excessively damage or wash away such depressions. After heavy rainfalls, inspect slopes for signs of erosion. Any such features should be repaired promptly by filling the eroded areas slightly above the original grade, re-roughened, and reseeded (or mulched). Continued monitoring is essential, especially on seeded slopes, to ensure vegetation is taking hold and that the roughened surface remains effective. For slopes 3H:1V or steeper, highly erodible slopes, or slopes where surface roughening may not provide adequate protection for other reasons, benching or terracing (Section 4.3.1) may be a more ideal measure.


Source: TDEC

Planning and Design Considerations

Though smooth, hard surfaces may be aesthetically pleasing on slopes, such surfaces increase erosion potential. Thus, rough slope surfaces are preferred. Rough and loose surfaces promote water infiltration, speed up the establishment of vegetation, and decrease runoff velocity while also providing natural coverage for lime, fertilizer, and seeds. Various roughening options exist, including grooving, tracking, and ripping. When selecting a surface roughening technique, consider cost, slope steepness, mowing requirements, and whether the slope is formed by cutting or filling.

Cut slopes for areas that will not be mowed

- Grooving may be implemented when stair-stepping is not necessary. Use machinery to create a series of ridges and depressions that run along the slope contour. Grooves may be constructed with discs, tillers, spring harrows, or the teeth on a front-end loader bucket. Such grooves should not be less than 3 inches deep or further than 15 inches apart (Figure 4.3.8-A).

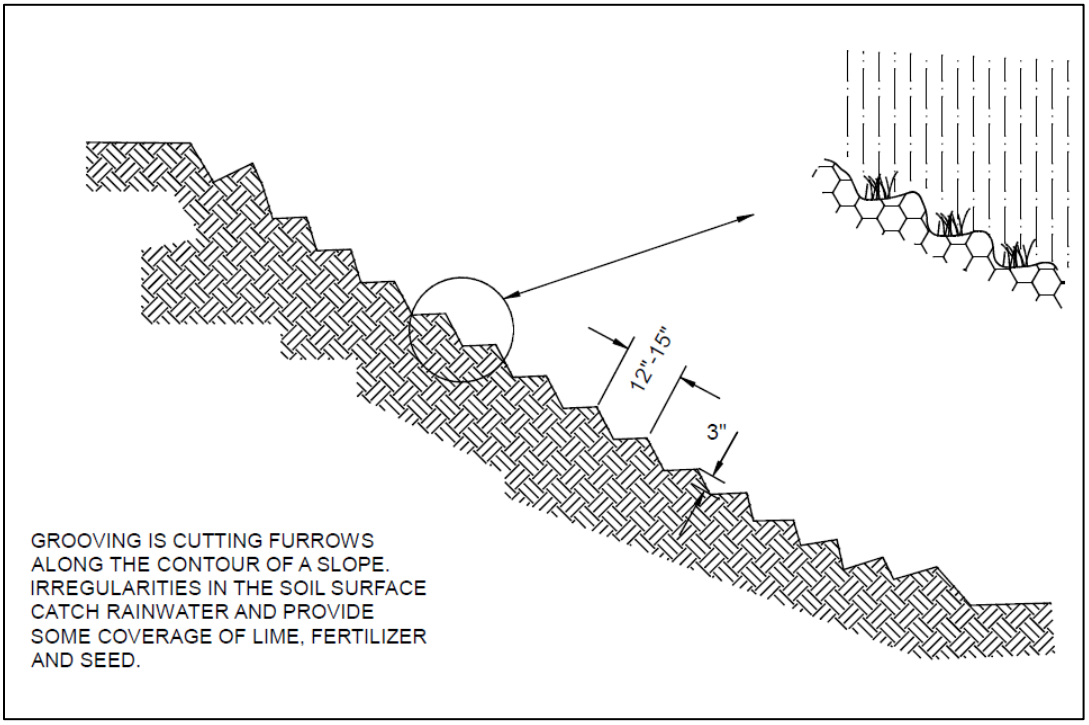


Figure 4.3.8-A: Grooving details. Adapted from VDEQ (2024).

Fill slopes for areas that will not be mowed

- Place fill slopes with a gradient steeper than 3H:1V in lifts not to exceed nine inches, and make sure each lift is properly compacted. Ensure that the face of the slope consists of loose, uncompacted fill four to six inches deep; and
- Do not blade or scrape the final slope.

Cuts, fills, and graded areas that will be mowed

- Mowed areas should not exceed slopes of 3H:1V;
- Roughen areas to shallow grooves by normal tilling, disking, harrowing, or use of cultipacker-seeder. Make the final pass of any such tillage implement on the contour;
- Make grooves, formed by such implements, close together (less than 10 inches) and not less than one inch in depth; and
- Avoid excess roughness in mowed areas.

Roughening with tracked machinery

- In general, tracking is not as effective as the other roughening methods;
- Limit roughening with tracked machinery to sandy soils to avoid undue compaction of the soil surface;

- Operate tracked machinery up and down the slope to leave horizontal depressions in the soil perpendicular to drainage. Do not back-blade during the final grading operation (Figure 4.3.8-B); and
- Promptly seed and mulch roughened areas for quick seed germination and growth.

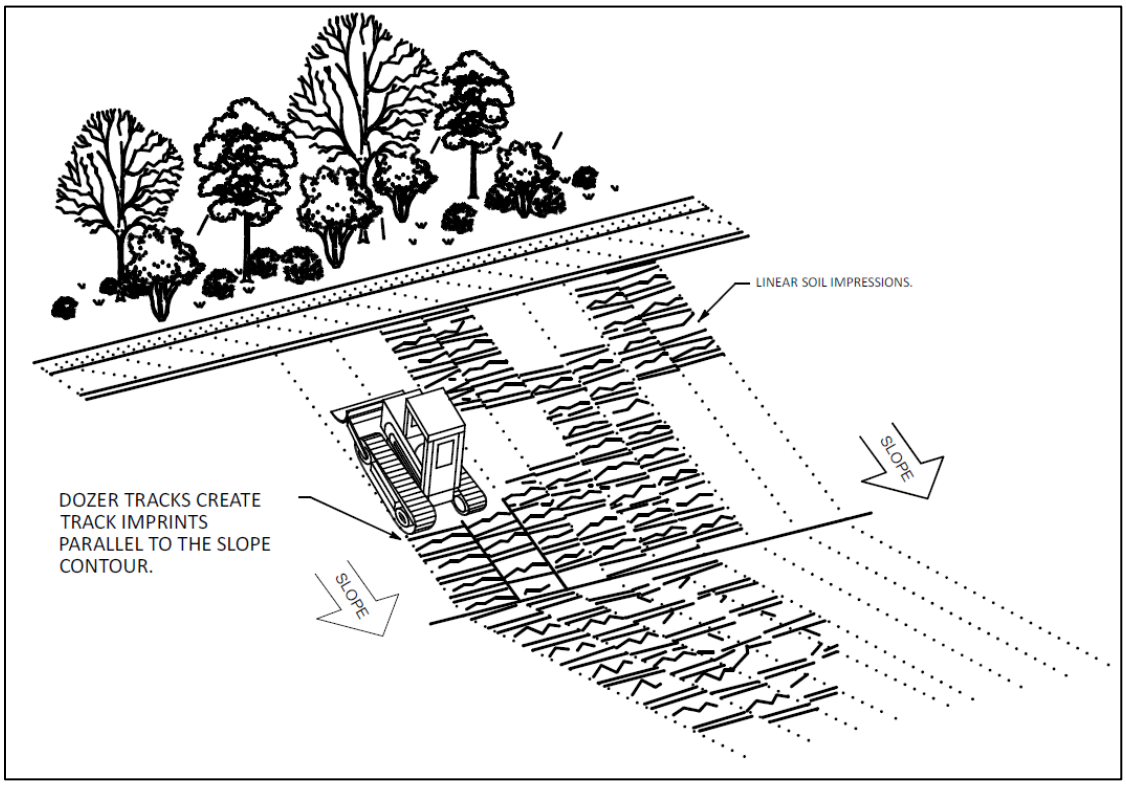


Figure 4.3.8-B: Tracking details. Adapted from VDEQ (2024).

Example Application

No formal design or quantities are required for this measure and therefore are not presented herein.

References

KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosions, Sediment, and Pollutant Runoff from Construction Sites.*
NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.
USEPA. (2021). *Stormwater Best Management Practices: Soil Roughening*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.4 Sediment Control

Sediment control practices attempt to prevent sediment that has already been eroded from leaving the construction site via stormwater runoff. These practices are generally installed at the most downgradient locations of a construction site and are therefore one of, if not the, last line of defense in mitigating sediment pollution. Engineers and construction personnel cannot solely rely on sediment control measures to comply with regulations. While sediment control measures can be incredibly efficient at trapping sediment onsite, the best way to prevent sediment impairment is for sediment not to be eroded and transported in the first place. The Tennessee CGP requires maintenance of all sediment control measures once 50% of the sediment storage capacity is reached. However, if maintenance is needed and recent rainfall has led to saturated site conditions, it may be acceptable to postpone maintenance operations for some measures as this could inadvertently contribute to erosion and sediment loss. When postponing maintenance operations, consult the engineer to understand if the remaining 50% capacity is sufficient for any imminent rainfall. Additionally, it is best to install supplemental sediment control measures that do not require heavy machinery such as wattles, filter socks, slash mulch berms, or silt fence, especially if site conditions are expected to remain saturated for an extended period of time because saturated conditions still must comply with all requirements in the CGP. By capturing eroded soils, these measures reduce water pollution, protect aquatic ecosystems, and maintain the integrity of stormwater drainage systems. However, there must be a plan in place for captured sediment once removed from the sediment control measure. Soils that have been eroded once, will surely erode again. It is typically encouraged for collected sediment to be moved to a location distant from concentrated flow paths (whether it be onsite or offsite) and stabilized.

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4.4.1 Brush Fabric Barrier



Source: ALSWCC (2018)

Definition and Purpose

A brush fabric barrier is a dam-like structure constructed from woody residue and faced with a geotextile fabric to function as a dam. The dam temporarily ponds water, allowing sedimentation. It is intended to function similarly to a sediment trap for a short period of time.

Appropriate Applications

This practice is applicable in small drainage areas (ideally less than two acres) and with a lifespan of less than one year (ALSWCC, 2018). Brush fabric barriers are ideal where brush and other woody debris are available from a clearing and grubbing operation.

Limitations and Maintenance

Brush fabric barriers are subject to several limitations and require regular maintenance to function effectively. They are only suitable on sites where topography allows for proper impoundment of sediment-laden runoff and where an adequate supply of woody material is available from clearing operations. If these conditions are not met, alternative sediment control measures may need to be considered.

Maintenance involves frequent inspections, particularly after significant rainfall, to check for short-circuiting or flow bypassing the ends of the barrier. Accumulated sediment should be

removed when it reaches half the height of the barrier to prevent overtopping or structural failure. Overtopping during large storm events can lead to gully erosion behind the barrier, which must be promptly repaired. Since brush fabric barriers are temporary, they should be removed once construction activities cease and the contributing area is stabilized through seeding and mulching unless otherwise specified in the site plan (MDEQ, 2011).

Planning and Design Considerations

Brush fabric barriers should be positioned downgradient from areas producing sediment-laden runoff and located in areas that provide adequate storage volumes, detention times, and that would not result in property damage or loss of life under sudden failures. The barrier is best used in areas with a maximum drainage area of two acres, and its design lifespan is approximately one year. When drainage areas exceed two acres, consider treatment trains with other EPSC measures such as additional brush fabric barriers (if multiple flow paths exist), sediment traps (Section 4.4.8), or sediment basins (Section 4.4.7).

Brush fabric barriers are constructed by placing densely compacted woody debris in a continuous row, generally along the contour of the land. Each end of the barrier is to be turned upslope to ensure that high flows overtop the barrier rather than bypassing it, similar to the “J-hook” concept that is applied to silt fence (Section 4.4.10). The brush should be oriented lengthwise along the barrier, with main stems aligned and smaller limbs trimmed to prevent puncturing the facing fabric. The lengthwise orientation allows longer pieces of brush to lock together and hold their shape. Barrier dimensions typically range from three to six feet in height, with a base width of at least five feet perpendicular to flow direction. The upstream face of the brush is typically covered with a non-woven geotextile fabric, such as an eight-ounce material (Figure 4.4.1-A). To maintain structural integrity, fabric rolls should be wide and long enough to minimize vertical splices and eliminate horizontal ones. Where vertical seams are necessary, a three-foot overlap should be securely fastened to prevent flow from short-circuiting. The base of the fabric is to be embedded at least six inches deep in an excavated trench in front of the barrier, staked securely at three-foot intervals using 18-inch wooden stakes, then backfilled and compacted. The top edge of the fabric should be anchored, typically with twine tied to stakes behind the barrier, to prevent sagging below the designed storage elevation. This ensures effective sediment containment and maintains structural stability during storm events. The barrier is to be removed once construction is complete and the drainage area is stabilized. The sediment accumulations should be properly stabilized prior to completion of the construction project.

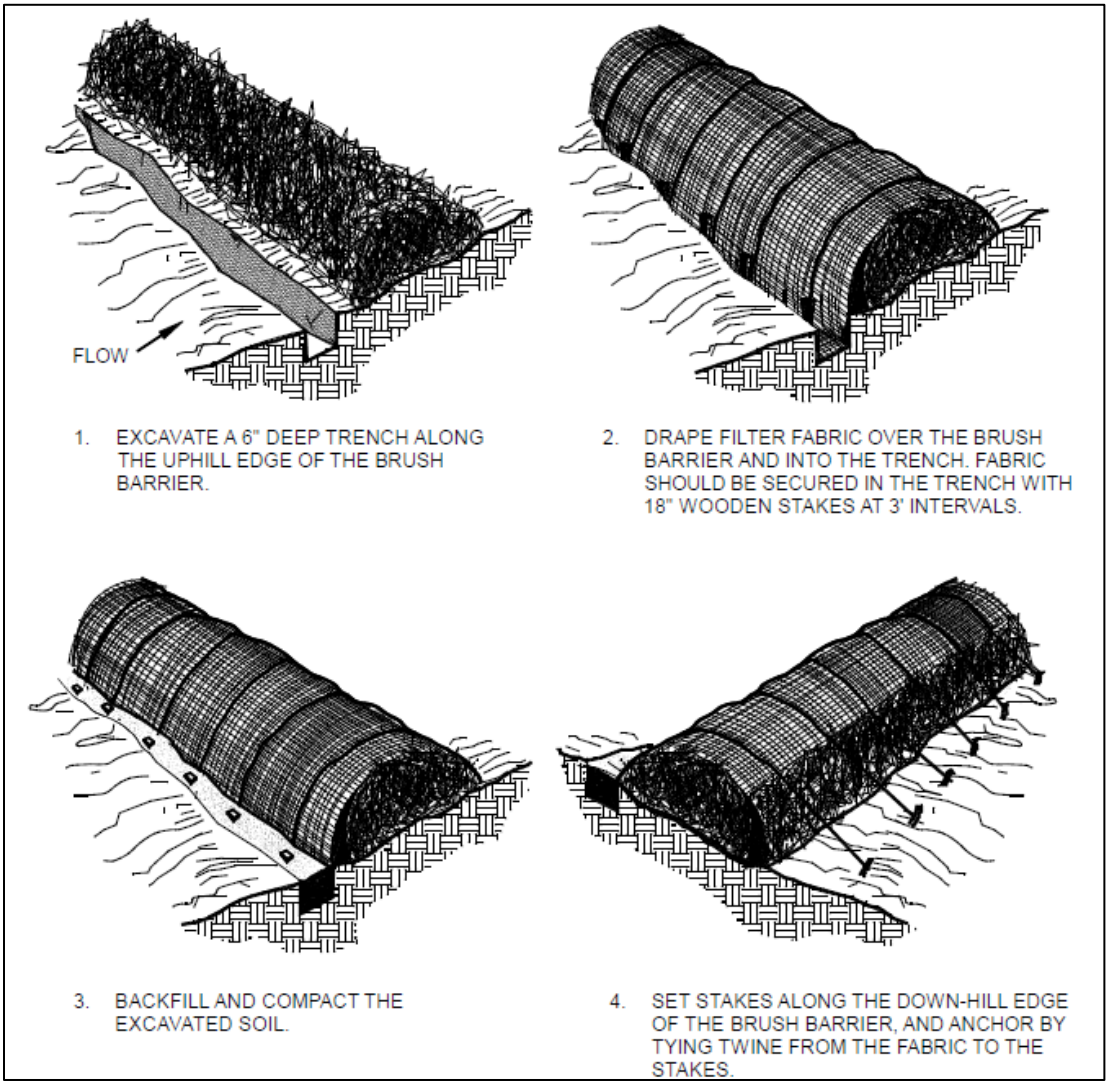


Figure 4.4.1-A: Typical brush fabric barrier construction details. Adapted from VDEQ (2024).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

MDEQ. (2011). *Mississippi Handbook for Erosion Control, Sediment Control, and Stormwater Management on Construction Sites and Urban Areas.*

VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.4.2 Filter Berms: Slash Mulch



Source: Young (2021)

Definition and Purpose

Slash mulch berms are linear, trapezoidal-shaped sediment control structures constructed from byproducts of land clearing operations (branches, topes, stumps, roots) that have been shredded (not chipped) into fibers six to 12 inches in length. These berms are strategically placed perpendicular to overland sheet flow on construction sites to trap sediment, reduce overland flow velocity, and may contain compost to enhance pollutant removal through biological and chemical filtration processes.

Appropriate Applications

Slash mulch berms serve as an alternative to traditional sediment barriers such as silt fence (Section 4.4.10) and sediment tubes and socks (Section 4.4.9). They are to be applied on contour along sloped areas or the (downgradient) perimeter and are applicable in sheet flow applications only. Slash mulch is particularly ideal in areas where staking and trenching are impractical, such as rocky or wooded slopes, and where land clearing operations occur onsite, such that materials do not need to be hauled in.

Limitations and Maintenance

Mulch is to be shredded, not chipped, as chipped mulch lacks the ability to lock together and hold its shape when formed into a berm (Roche, 2023). Even so, slash mulch berms are less effective in large drainage areas, on steep slopes, or in areas of concentrated flow (Iowa

SUDAS, 2019; USEPA, 2021). TDOT recommends a maximum slope length of 300 feet for any single berm. Routinely inspect filter berms and maintain them to a functional condition throughout construction. Slash mulch berms trap sediment within and upgradient of the measure (Roche, 2023), reducing the sediment removal maintenance burden. However, if or when sediment accumulates behind the measure to half the height of the structure, remove the sediment and dispose of it or stabilize it according to the plans.

Planning and Design Considerations

Filter berms are used as sediment barriers and are to be placed on slope contours, six to 10 feet away from the toe of a slope, or along the downgradient perimeter of the construction site. When they cannot be placed on contour, and it is determined that a berm is still an ideal EPSC measure, they can be formed into "J-hooks" which is a common design modification to silt fence (Section 4.4.10). Filter berms are not to be placed in floodplains, stream buffers, channels, or other areas of concentrated flow. TDOT suggests that a single filter berm should not have a drainage area exceeding one-quarter acre per 100 linear feet of berm, and upgradient length should be limited depending on slope steepness (Table 4.4.2-A). When slope lengths are exceeded, use multiple berms. Some agencies provide guidance suggesting Table 4.4.2-A is conservative (VDEQ, 2024), while others would suggest the opposite (Iowa SUDAS, 2019). Filter berms are an ideal sediment barrier alternative as they are low cost (i.e., can be constructed from onsite material, unless compost is included in the design), low maintenance, environmentally friendly, wildlife friendly, and because they naturally biodegrade, can be left onsite after project completion. In more radical scenarios, filter berms may require additional structural components, such as silt fence, wattles, nettings, to prevent mulch displacement; however, this may negate some of the advantages of this measure, such as the ability to leave it in place after project completion and its environmental- and wildlife-friendly aspect. If compost is to be included in the mulch mixture, it should be sourced from an approved supplier. Ultimately, the engineer is responsible for designing a filter berm that is effective and remains in place, while considering the cost, availability of materials, and its environmental/wildlife-friendly status.

Table 4.4.2-A: Maximum suggested upgradient lengths for filter berms before multiple should be considered. Source: TDOT.

Slope (%)	Upgradient Length (ft)
< 2	110
2 - 5	100
5 - 10	75
10 - 20	25
>20	Not Recommended

Typical filter berms are linear, trapezoidal-shaped structures with a bottom width of four to seven feet, a height of one to three feet, a top width of at least two feet, and ends of the berm turned upward, limiting bypass (Iowa SUDES 2019; TDOT; USEPA, 2021; VDEQ, 2024). Compost should contain organic matter combined with slash mulch. In order for a compost filter berm to function as intended, only compost products that meet all federal regulations are permitted for use. Therefore, products certified by the United States Composting Council's Seal of Testing Program (www.compostingcouncil.org) are recommended as specified in Section 4.2.6.2. Filter berms absent of compost are not intended to support vegetation; therefore, they should not be seeded. A compost filter berm can support vegetation, if desired, and if the material is properly graded and the portion of compost is 65% or less (TDOT). A compost filter berm that has been seeded and stabilized can be left in place as a permanent feature, while an unvegetated compost berm or mulch filter berm can be either removed or spread on the site at the discretion of the engineer.



Roche (2023) investigated the efficacy of slash mulch berms and concluded that a berm constructed with Nebraska DOT specifications (trapezoidal shape, six-foot width at base, two-foot width at top, three feet in height, and slash mulch sourced from central Alabama) retained 61% of sediments onsite. Two simple alterations of the Nebraska DOT specifications M1 (three-foot width, one and a half foot height, compacted in three six-inch lifts with a jumping-jack compactor) and M2 (six-foot width, one foot height, compacted in two six-inch lifts) resulted in 73.5% and 78.2% of total sediment retention, respectively. Under the exact same testing conditions [refer to Roche (2023) for specific methodologies], Nebraska DOT straw wattle and silt fence applications retained 62.5% (77.2 – 80.8, depending on design modifications) and 83% (66 – 86, depending on design modifications) of sediments, respectively. These results suggest innovative measures such as filter berms that can be freely sourced from onsite materials; require less person-hours for construction, maintenance, and removal; require less materials (e.g., no staking); and are environmental- and wildlife- friendly can achieve similar, and in some cases, better sediment



retention compared to more traditional sediment barriers that may not be environmentally friendly.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

Iowa SUDAS. (2019). *Erosion and Sediment Control*.

MPCA. (2023). *Minnesota Stormwater Manual: Sediment control practices –Stabilized earth/soil berm.*

Roche, B. G. (2023). *Performance Evaluation of Sediment Control Practices* (Master's thesis, Auburn University).

TDOT. *Drainage Manual Ch10*.

USEPA (2021). *Stormwater Best Management Practices: Brush Barrier*.

VDEQ. (2024). *Virginia Stormwater Management Handbook.*

Young, C. (2021). *Mulch Berm Sediment Barrier*. Alabama Power Company.

4.4.3 Filter Strip



Source: INDOT (2025)

Definition and Purpose

A filter strip is a uniformly graded, densely vegetated area designed to intercept sheet flow from adjacent disturbed or impervious surfaces on construction sites. Vegetation may consist of preserved existing cover or newly planted species established to form the filter strip, which functions to remove sediment, absorb other pollutants, and promote infiltration as sheet flow passes through the measure.

Appropriate Applications

Filter strips are only applicable in sheet flow conditions and are to be used in combination with other EPSC measures. They are commonly implemented at the toe of slopes in addition to silt fence or wattles, downgradient of impervious areas, and downgradient of level spreaders (Section 4.3.5) that are used to create unconcentrated runoff.

Limitations and Maintenance

Filter strips lose efficacy when the contributing slopes are overly steep (approximately 5% or greater) or contributing slope lengths are excessively long (approximately 100 feet or greater) (Dane County, 2025). Further, filter strips are not effective measures in concentration flow scenarios.

The filter strip vegetation must be maintained to remain effective, and if planting new seedlings, appropriate species, planting rates, and planting timelines must be followed. Vegetation should be inspected regularly, particularly during establishment, to ensure healthy growth. Re-seeding, watering, fertilizing, or trimming should be performed as

necessary. During inspections and following large storm events, check to identify issues such as rill or gully formation, bare spots, and excessive sediment accumulation, all of which indicate the system may not continue to function as intended without maintenance. Mow using lightweight equipment and remove accumulated sediment and litter as necessary (Dane County, 2025; KTC, 2015).

Planning and Design Considerations

Filter strips are most effective when installed early in the construction process, ideally before general land disturbance begins, or when existing vegetation that meets density and coverage requirements can be preserved. These strips should be strategically placed on contour (i.e., parallel to contour lines) and immediately downslope of disturbed areas to intercept sheet flow before it reaches environmentally sensitive areas. Concentrated flows must be dispersed, often using a level spreader (Section 4.3.5) or other energy dissipators (Section 4.3.4), before entering the strip to preserve uniform sheet flow and prevent channelization. Dimensions of the filter strip are to be tailored to site specific factors such as contributing drainage area size, slope steepness, sediment load, and vegetation characteristics. For effective sediment removal, slopes above the strip should generally range from 1% to 5%. Filter strips will lose efficacy when placed downgradient of slopes greater than 5% and are not recommended when slopes exceed 10%. Drainage area-to-strip ratio should remain below 10:1 (ALSWCC, 2018). Widths (perpendicular to the direction of flow) vary depending on adjacent land use; KTC (2015) recommends minimum widths of 25 feet for urban streams and up to 100 feet for large rivers. Widths less than 15 feet will likely be ineffective.

If the filter strip is to consist of preserved vegetation, indicate so on the plans and ensure the designated area is fenced or flagged so as not to be disturbed. If planting new vegetation (or sod), the area should be prepped before planting. Prep the seedbed and apply soil amendments using best practices as described in Section 4.2.6.11 and as determined via soil tests. The ground is to be uniformly sloped perpendicular to the direction of flow.

Vegetation in the filter strip must consist of dense (>90% coverage), stiff-stemmed herbaceous species capable of filtering particles (Dane County, 2025). Use information from the soil test or the Seed Mix Appendix (Appendix D) to select the appropriate species, ensuring it is tolerant to frequent inundation, extended dry periods, and matting. Protect the seedlings with adequate mulching (Section 4.2.6.9) practices.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

Dane County. (2025). Dane County Stormwater Manual: Filter Strip.

KTC. (2015). *Best Management Practices (BMPs) for Controlling Erosion, Sediment, and Pollutant Runoff from Construction Sites*.

INDOT. (2025). *INDOT Construction Stormwater Management Field Guide.*

4.4.4 Inlet Protection: Area



Source: TNWRRC

Definition and Purpose

Area inlet protection is a temporary measure installed around, on top of, or inside storm drain drop inlets or catch basins to prevent sediment from entering the storm drainage system during construction. These structures create ponding around the inlet, allowing coarse sediments to settle and, in some cases, filtering finer particles from runoff before it enters the system.

Appropriate Applications

Area inlet protection is appropriate for use on operational drop inlets or catch basins that receive runoff from disturbed drainage areas, particularly before site stabilization has been achieved. Various constructed and manufactured types exist, each of which with unique characteristics. Specific conditions where one type may be more appropriate than another are discussed in the planning and design criteria section.

Limitations and Maintenance

Inlet protection devices will likely cause ponding and, therefore, are not suitable for high-traffic areas or locations where standing water could create hazards. These measures are considered secondary sediment controls and should always be used in conjunction with additional upstream EPSC measures, as they are not capable of filtering out fine silt and clay particles or effective under high sediment loads. A common limitation is the tendency for materials such as stone, gravel, or filter fabric to clog quickly, which can lead to bypass flows,

increased ponding, and even erosion of unprotected areas if stormwater cannot enter the system. Maintenance demands are high compared to other measures, with sediment often needing to be removed after each storm event to restore capacity. Further, inspect the device to determine if it is structurally sound and will not collapse under sediment or hydrostatic loads. Additional upkeep may involve inspecting downstream structures after severe storms for bypassed sediment, cleaning accumulated debris from adjacent areas, and ensuring inlet protection devices and accumulated sediment are removed and disposed of properly after site stabilization (TDOT). Silt fence is not an approved inlet protection measure unless augmented with structural innovations as described in Section 6.4.

Planning and Design Considerations

All area inlet protection devices are to be installed before soil disturbance occurs within the drainage area, and in new developments, they should be installed immediately after the storm sewer inlets are constructed. There are various types of area inlet protection practices that can be constructed from common materials such as conventional riprap, sandbags, wattles, hardware cloth, etc., in addition to manufactured devices that are commercially available. Commercially available products are viable options that are structurally supported to withstand sediment and hydrostatic loads, pond water, and allow sediments to settle, do not float or undermine, do not cause erosion of the soil surface between the device and inlet, and have a dewatering mechanism that prevents onsite flooding during large rain events (ALSWCC, 2018).

Excavated drop inlet protection is designed for locations where relatively heavy flows are expected, and overflow capacity is needed. The practice involves excavating a temporary basin around the inlet to create storage volume that allows sediment to settle before runoff enters the storm drainage system (Figure 4.4.4-A). Basins are typically one to two feet deep, measured from the top of the inlet structure, with side slopes no steeper than 2H:1V. Storage capacity should be sized at approximately 134 cubic yards (about 1,800 cubic feet) per acre of contributing drainage area, and the basin should be shaped to fit site conditions. For maximum trapping efficiency, the longest dimension of the basin is oriented in the direction of inflow, and rectangular basins are recommended where concentrated flows occur, such as in highway medians. Construction begins by clearing debris, grading the approach to the inlet uniformly, and protecting any weep holes with gravel. The basin should include provisions to drain temporary pools to improve sediment trapping and reduce problems from standing water after storms. Regular maintenance is essential: sediment should be removed once it reaches half the design depth, and the trap restored to its original dimensions with proper disposal of spoil to a stable location. When the contributing drainage area has been permanently stabilized, the basin is decommissioned by sealing weep holes, backfilling to grade with compacted stable soil, and establishing final site stabilization.

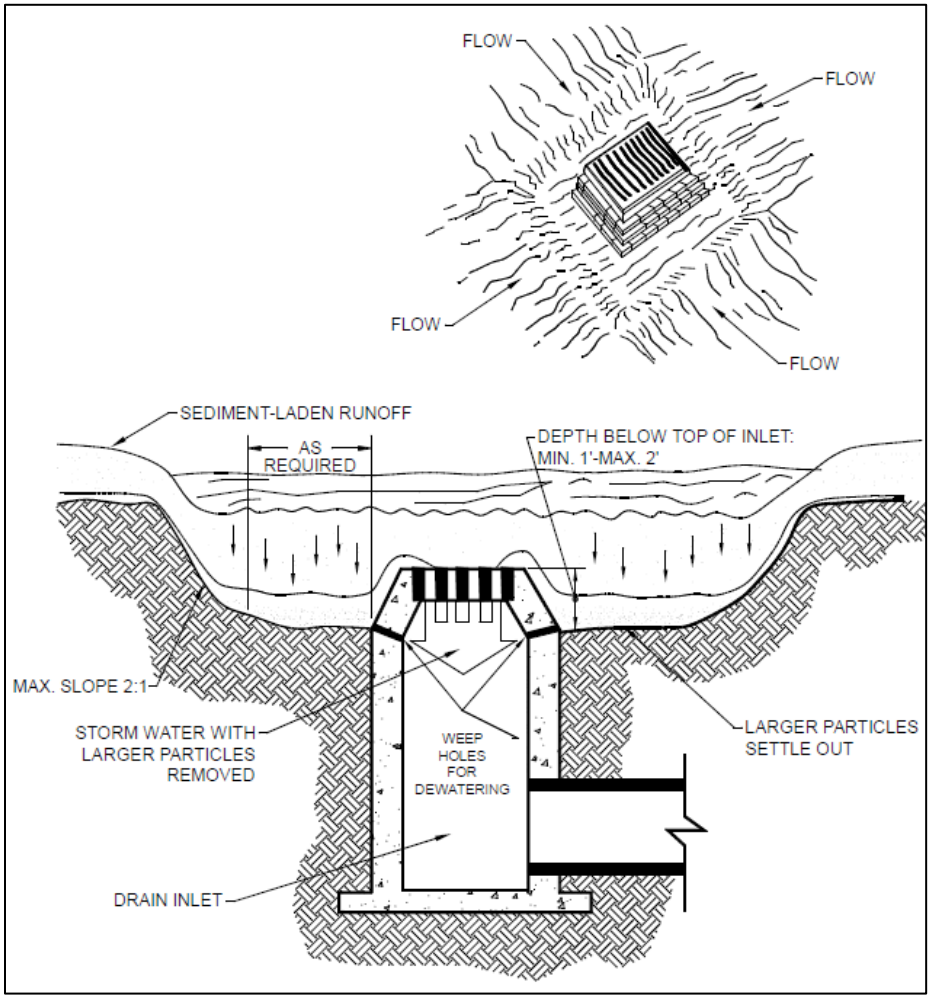


Figure 4.4.4-A: Excavated drop inlet protection details. Adapted from VDEQ (2024).

Hardware cloth and gravel inlet protection is designed for catch basins or drop inlets that receive light to moderate flow, particularly where shallow sheet flow is expected. This practice is most effective in relatively flat areas with slopes of less than one percent, and it should not be used near the edge of fill material or in locations where water could be diverted over cut or fill slopes. Each protected inlet should drain no more than one acre, ensuring proper function and sediment capture. The structure consists of a wire mesh barrier supported by steel T-posts and surrounded by clean gravel, forming a stable sediment trap that allows storm flows to pass through the inlet



rather than bypassing the structure (Figure 4.4.4-B). For construction, five-foot steel T-posts (1.25 pounds per linear foot) are driven at least two feet into the ground, spaced no more than four feet apart around the inlet. A minimum 19-gauge hardware cloth with 0.25-inch openings is secured to the posts at the top, middle, and bottom, with a two-foot flap extended on the ground and anchored beneath the gravel. Clean stone, such as #5 or #57 gravel, is placed on a 2:H1V slope to a height of roughly 16 inches against the hardware cloth, providing both structural stability and filtration. The top of the gravel structure should remain at least 12 inches lower than the downslope ground elevation to ensure that all flows pass through the inlet rather than bypassing it. Temporary dikes may be constructed below the inlet to further prevent bypass. Once the drainage area is stabilized, accumulated sediment should be removed, the barrier dismantled, and the site properly compacted and stabilized with groundcover.

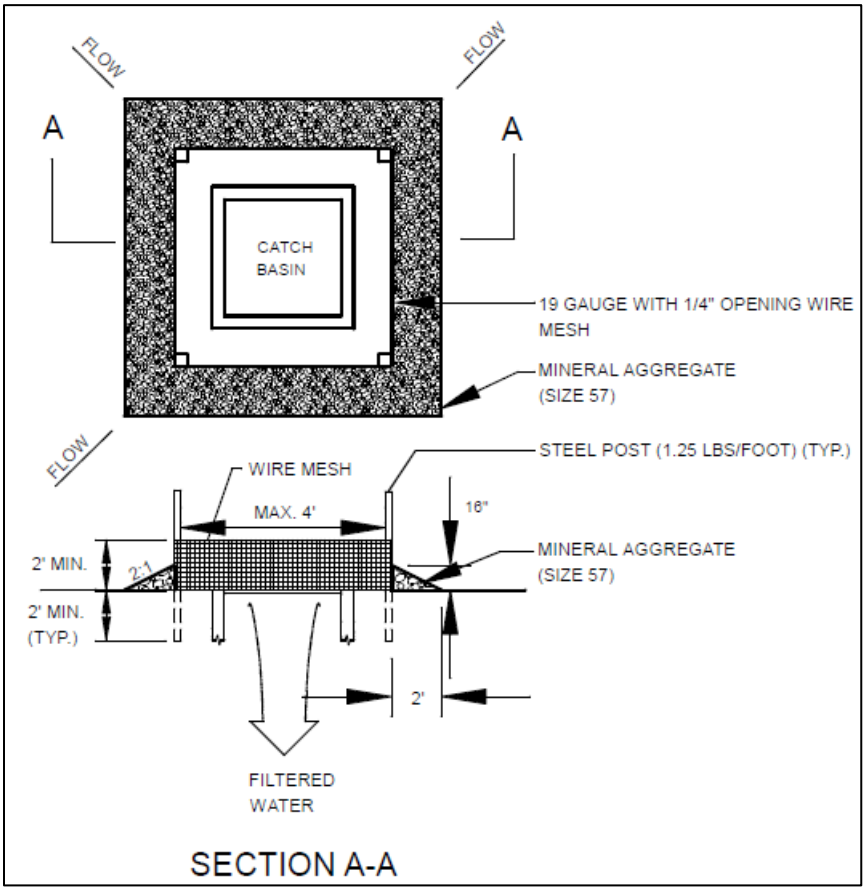


Figure 4.4.4-B: Hardware cloth and gravel filter inlet protection details. Source: TDOT.

Block and gravel inlet protection is designed as a temporary practice used at storm drain drop inlets where heavy flows are anticipated, and overflow capacity is needed to reduce

excessive ponding. The method consists of placing concrete or cinder blocks lengthwise on their sides in a single row around the inlet, with holes oriented horizontally and the ends tightly abutting. The barrier height should be no less than 12 inches and no greater than 24 inches, with the first course recessed at least two inches below the storm drain crest for stability. Additional courses may be stacked in a staggered pattern, with lateral support provided by wood studs if needed, and some blocks laid sideways at the lowest elevation to serve as dewatering points. Wire mesh or hardware cloth with half-inch openings is fitted over all block openings to prevent gravel loss. Clean stone, such as #57 washed stone, is then placed against the blocks in a triangular cross-section, extending to the top of the barrier and sloping outward at 2H:1V or flatter. A geotextile underlayment may also be installed beneath the blocks, gravel, and immediately around the inlet to provide additional stability and filtration. The top elevation of the structure should remain at least six inches below the adjacent downslope ground to ensure stormwater passes through the structure and into the drain rather than bypassing it. While this practice effectively reduces sediment entry into storm drains, shallow ponding following rainfall is expected, and accumulated sediment should be frequently removed to maintain performance.

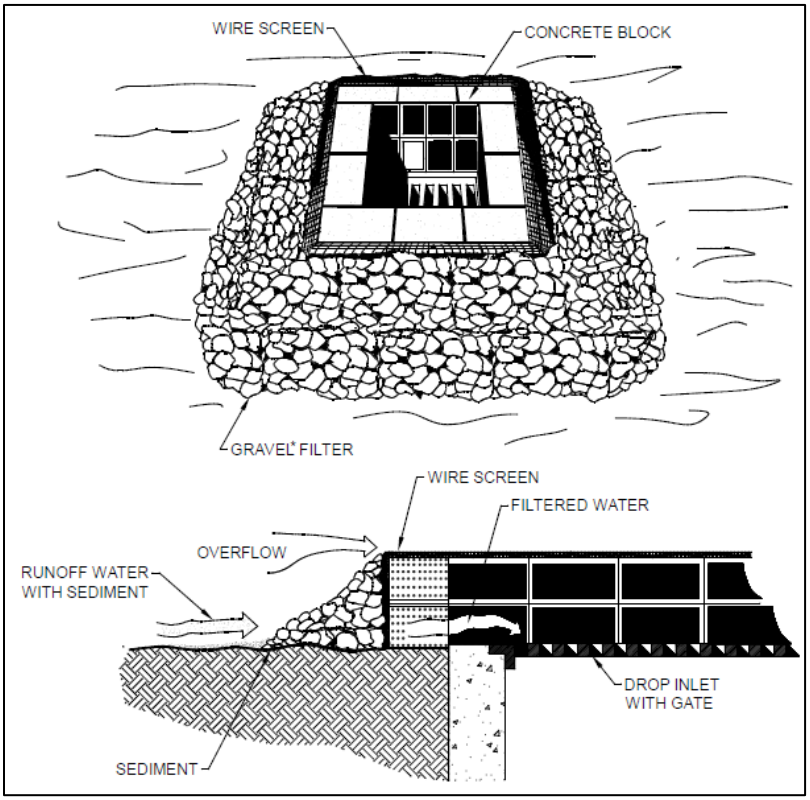


Figure 4.4.4-C: Block and gravel inlet protection details. Adapted from VDEQ (2024).

Rock ring inlet protection is designed for drop inlets that receive high velocity stormwater flows or drainage from larger contributing areas. The practice captures sediment within an excavated depression surrounding the inlet while reducing flow velocities before water enters the storm drain (Figures 4.4.4-D). A one-foot-wide level area, recessed four inches below the crest of the drop inlet, provides added protection against sediment intrusion. Conventional riprap, typically Class B, is placed in a ring around the inlet (ideally on top of a geotextile fabric to prevent piping) with side slopes no steeper than 2H:1V, a crest width of at least 18 inches, a bottom width of seven and a half feet or greater, and a height ranging from two to three and a half feet. The outside face of the riprap is wrapped in a 12-inch layer of clean, washed stone such as #57 aggregate, which enhances filtration and prevents fine sediment migration. Wire mesh with two-inch openings may also be placed over the inlet grate to intercept debris, but it needs to be inspected regularly to prevent clogging. The top elevation of the rock ring should remain at least 12 inches below the adjacent downslope ground surface to ensure stormwater passes through the structure and into the drain rather than bypassing it. In some cases, temporary dikes constructed downslope may be necessary to prevent uncontrolled flow around the structure. Construction involves clearing debris, grading a shallow depression toward the inlet with stable side slopes, installing the riprap to the specified dimensions, and backfilling with washed stone. Periodic sediment removal is required to maintain storage capacity and hydraulic function.


Source: TDEC

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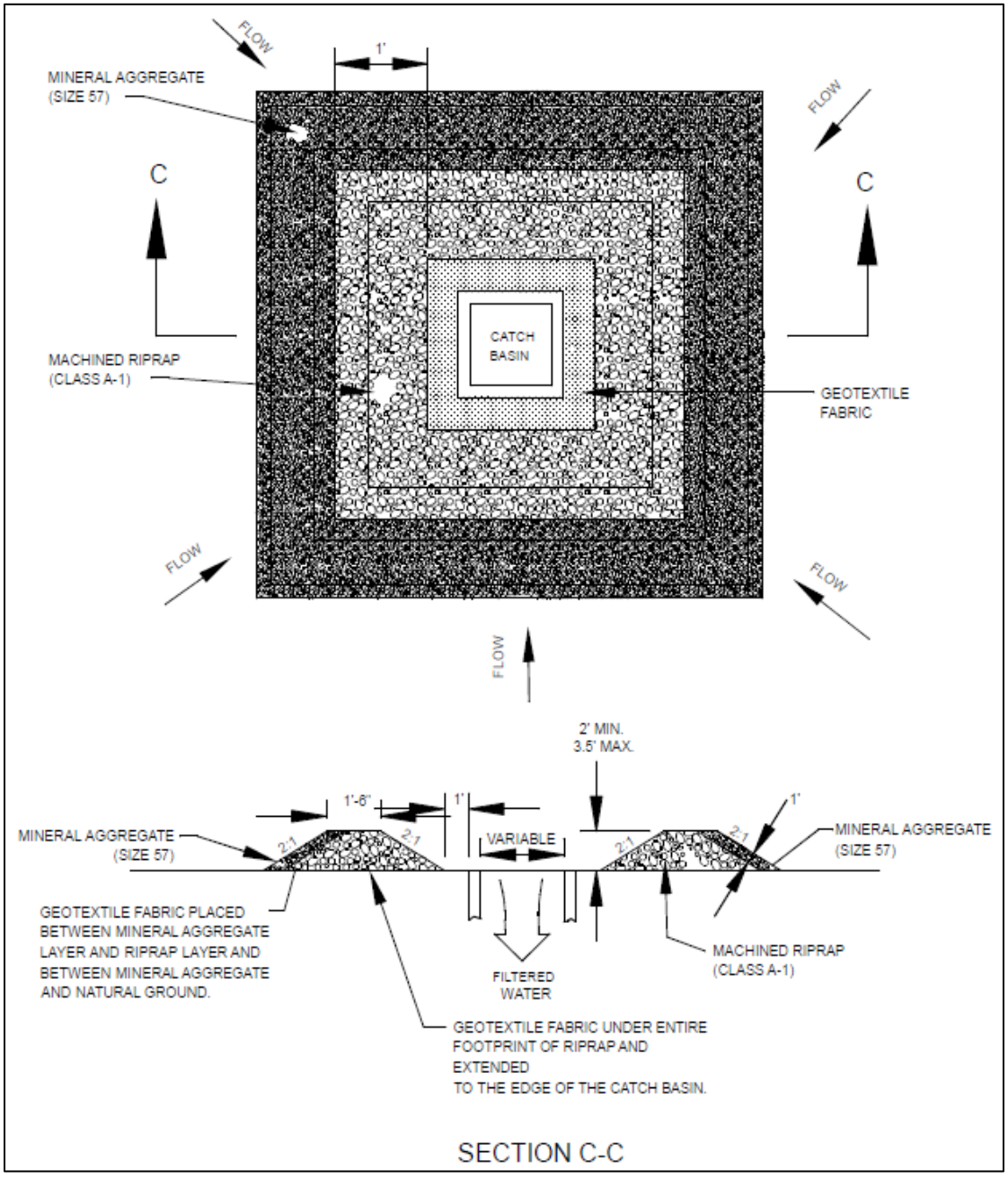


Figure 4.4.4-D: Rock ring inlet protection details. Source: TDOT.

Sandbags and wattles can both serve as effective inlet protection devices when properly installed around stormwater drop inlets (ALSWCC, 2018). In both applications, an eight-ounce non-woven geotextile is first placed as an underlayment on compacted earth, extending at least one foot beyond the perimeter of the barrier and pinned securely at the inlet and edges. Both methods involve extending the barrier at least one foot away from the inlet, ensuring space for ponding and sediment capture before runoff enters the drainage system. Despite these similarities, the materials differ in their construction and installation. Sandbag protection relies on stacking multiple layers of bags in a triangular cross-section to achieve stability, with the bags placed tightly together (often in a staggered pattern) to form a sealed perimeter. In contrast, wattles require overlapping ends by at least 18 inches and securing them with stakes or sod staples to prevent movement or flotation, with denser wattles preferred to promote ponding. While sandbags achieve strength through layering and compaction, wattles rely on staking and density for performance, but both serve the common purpose of slowing stormwater and trapping sediment at drop inlets. <u>*Do not stake through the straw wattles,*</u> instead, form an A-Frame to leave the wattle intact and prevent floatation.


Source: TNWRRC


Source: TNWRRC

There are several manufactured devices that are listed on TDOT's QPL report. A number of popular options include:

- Silt Saver Inlet Protection Products;
- Drain Filter for Oily Runoff & Sediment - FLT116 - New Pig;
- Drop Inlet Protection Filter — Mainline Materials; and
- Marsh Mat - Storm Inlet Filter Pad Kit - The Drainage Products Store.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

TDOT. *Drainage Manual Ch10*.

4.4.4.1 Catch Basin Filter Assemblies





Source: TDOT

Definition and Purpose

Catch basin filter assemblies are temporary sediment control structures installed over catch basins and designed to prevent suspended solids from entering a storm drainage system. They are typically constructed with a lumber frame covered in hardware cloth and geotextile fabric (wire mesh can be used to help support the fabric). The geotextile fabric, which fully encloses the frame and is trenched into the surrounding ground, creates a complete barrier around the catch basin, ensuring all runoff passes through the filtration system. In addition to removing sediment, these assemblies can reduce stormwater flow velocities, promoting sediment deposition upstream of the inlet (TDOT).

Appropriate Applications

Catch basin filter assemblies may be constructed around any catch basin, regardless of shape. They may be more ideal than inlet protection (Section 4.4.4) when drainage areas are larger, flow velocity is greater, and where overtopping of traditional inlet protection is anticipated.

Limitations and Maintenance

Catch basin filter assemblies likely decrease the hydraulic capacity of the inlet. Further, they will pool water behind the structure, so they must only be used where sufficient space for such pooling exists. Unlike most other sediment control measures, sediment and debris accumulation behind catch basin filter assemblies are to be cleared after every storm. The removed sediment must be stabilized onsite or disposed of properly in accordance with the plans.

Planning and Design Considerations

Formal design of this measure is not required; however, the maximum discharge for each catch basin filter assembly should be checked based on the maximum ponding depth at each location (Table 4.4.4.1-A). Where the flow rates at a catch basin site exceed the maximum values presented in Table 4.4.4.1-A, an alternate type of sediment control measure should be considered. At each catch basin filter assembly, the type of device should be labeled in the EPSC plans. The various types of catch basin filter assemblies are designed to fit specific types of catch basins. Refer to TDOT Drainage Manual Chapter 7 for a detailed description of the catch basin types listed. The associated TDOT example drawings describe the use of catch basin filter assemblies for round catch basins and details for each specific type (i.e. – size) of filter assembly.

Table 4.4.4.1-A: Catch basin filter assembly types applicable for specific catch basin types. Refer to TDOT Chapter 7 for detailed descriptions of each catch basin type. Assume 1.5 feet of head on the filter and a 50% clogging factor. Source: TDOT.

Filter Assembly Type	Catch Basin Types	Catch Basin Size (feet-in^2)	Allowable Peak Discharge (cfs)
1	12LP, 25LP, 28LP, 38S, 41LP, 42S	4	10
2	10SB, 12SB, 25SB, 38SB, 39S, 41SB, 42SB	5-4 or 48 in round	13
3	12SC, 25SC, 38SC, 39SC, 41SC, 42SC, 51SC	6-6 or 60 in round	15
4	12SD, 25SD, 31SD, 39SD, 41SD, 42SD, 51SD	8-4 or 72/84 in round	19
5	12SE, 14SE, 25SE, 31SE, 39SE, 41SE, 46SE	10-8 or 96/108 in round	24
6	10S, 12B, 12P, 12S, 13B, 13P, 13S, 25B, 25P, 25S, 28B, 28P, 28S, 41P, 41S	5-4 by 4-4	12
7	14B, 14P, 14S, 26P, 26S, 29P, 29S	9-4 by 4-4	16
8	16B, 16S, 27S, 40S, 43SB, 45S	9-4 by 6-6	17
9	17S, 43SC	9-4 by 6-6	18
10	Special (132 inch by 132 inch)	13	30
11	Special (144 inch by 48 inch)	13-4 by 5-4	21

Catch basin filter assemblies are constructed of filter fabric and hardware cloth over a lumber frame, with the option of wire mesh to reinforce the fabric. The fabric should be trenched in and secured with stone along the four sides of the base. Clogged or damaged fabric must be replaced, and the lumber frame reused.

When specifying the use of catch basin filter assemblies, careful consideration should be given to upstream constraints on the allowable ponding depth. The allowable discharges

provided in Table 4.4.4.1-A may need to be adjusted if the allowable ponding depth is less than one and a half feet. This is especially important when specifying filters for catch basin inlets located on a long, sloped median.

When utilizing these filters in series on a continuous slope, it may be necessary to provide some temporary means of pooling the water around the filter assembly in order to minimize the amount of flow that would bypass. This will help to maximize the flow captured by an inlet on grade and help to prevent flows at the most downstream catch basin from becoming excessive. This may be accomplished by means of check dams (Section 4.3.3) placed immediately downstream of the filters.

Example Application

-Example Courtesy of TDOT-

Given:

A Type 38S catch basin is to be located adjacent to a freeway median crossover. The inlet will collect runoff from an area of 1.912 acres, which ultimately drains to a known Exceptional Tennessee Water. Rational method computations for the design of the storm drainage system indicate that the runoff coefficient at this site is 0.6 and that the 5-year rainfall intensity is 6.3 inches per hour. In order to ensure that the roadway sub-base will be properly drained, the maximum allowable ponding depth at the site is 1.2 feet. Due to the sensitive nature of the receiving stream, a catch basin filter assembly is desired for this site in order to ensure the greatest possible filtration of sediments.

Determine:

The type of catch basin filter assembly required for this site, and check to ensure that the depth of ponded water will be less than allowable.

Solution:

Based on Table 4.4.1-A, a Type 1 catch basin filter assembly is selected for this site. In order to check the depth of ponding which would occur in the 5-year storm, it is necessary to compute the peak discharge, using the Rational Method (Eqn 9):

$$q_p = c \times i \times A = 0.6 \times 6.3 \times 1.912 = 7.23 \text{ cfs}$$

The maximum allowable flow rate at a Type 1 catch basin filter assembly for a ponding depth of 18 inches is shown in Table 4.4.4.1-A to be 10 cfs. Since the flow through the filter fabric on the assembly essentially corresponds to flow through a porous medium, it is possible to estimate the ponding depth associated with 7.23 cfs simply by taking the ratio of the flow rates. Thus:

$$\text{Ponding Depth} = 7.23 / 10 \times 1.5 = 1.1 \text{ ft}$$

Since the ponding depth at the 5-year flow rate will be less than 1.2 feet, the catch basin filter assembly may be used at this site.

References

TDOT. *Drainage Manual Ch7*.
TDOT. *Drainage Manual Ch10*.

4.4.5 Inlet Protection Curb



Source: City of Franklin

Definition and Purpose

Curb inlet protection controls are typically placed on or around curb inlets to intercept or filter runoff, reducing sediment, debris, and other pollutants from entering storm drains where they could clog pipes, diminish capacity, degrade water quality, and contribute to localized flooding. By slowing flow velocities near the inlet, these measures promote sediment settling and, in some cases, provide filtration before stormwater enters the drainage system (TDOT). Inlet protection is the last opportunity to treat runoff before it is discharged, and thus, the function and maintenance of these measures is highly important.

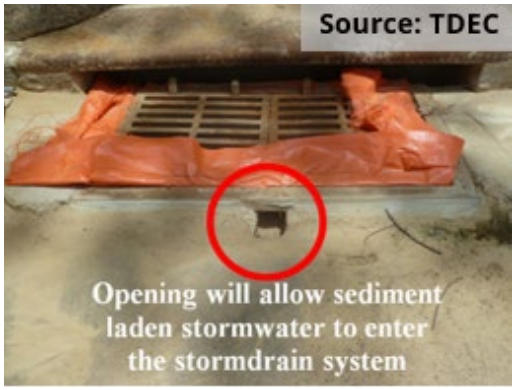
Appropriate Applications

Curb inlet protection is appropriate for use on operational curb inlets that receive runoff from disturbed drainage areas, particularly before site stabilization has been achieved. These measures are installed after the inlet grate and roadway surface are in place and can be applied to both on-grade and sag inlets by selecting the proper design (TDOT). Block and gravel systems are recommended for high flow conditions where overflow is likely, while fabric barriers may be used in smaller or flatter drainage areas, though they carry a higher risk of failure if not properly maintained (USEPA, 2021).

Limitations and Maintenance

Inlet protection devices will likely cause ponding and, therefore, are not suitable for high-traffic areas or locations where standing water could create hazards. These measures are considered secondary sediment controls and should always be used in conjunction with

additional upstream EPSC measures, as they are not capable of filtering out fine silt and clay particles or effective under high sediment loads. A common limitation is the tendency for materials such as stone, gravel, or filter fabric to clog quickly, which can lead to bypass flows, increased ponding, and even erosion of unprotected areas if stormwater cannot enter the system. Maintenance demands are high compared to other measures, with sediment often needing to be removed after each storm event to restore capacity. Additional upkeep may involve inspecting downstream structures after severe storms for bypassed sediment, cleaning accumulated debris from paved areas, and ensuring inlet protection devices and accumulated sediment are removed and disposed of properly after site stabilization (TDOT).



Planning and Design Considerations

All curb inlet protection devices are to be installed before soil disturbance occurs within the drainage area, and in new developments, they should be installed immediately after the storm sewer inlets are constructed. It is critical to consider all stages of construction. For example, when roads are installed to binder grade (i.e., not yet finalized), diversions are likely necessary to divert water from manholes (where water cannot yet enter due to the differences in grade elevation) to the curb. Generally, curb inlet protection measures/devices are not effective sediment retention devices and may result in flooding if drainage areas exceed one acre (TDOT; USEPA, 2021). When drainage areas exceed one acre, it is imperative that upstream EPSC measures detain, divert, or slow runoff from reaching the inlet.

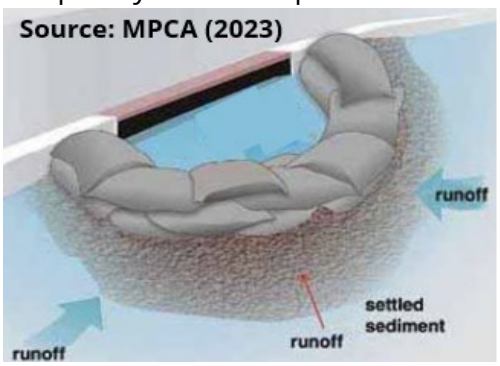
There are multiple types of curb inlet protection devices, each with unique characteristics and functionalities. The four types discussed herein include block and gravel (Type 1), sandbag or stone bag dam (Type 2), coarse aggregate wedge (Type 3), and filter fabrics (Type 4). In addition to the four constructed curb inlet protection devices, a variety of manufactured devices are available, such as dandy sacks, silt sacks, curb inlet filters, etc. TDOT maintains a comprehensive QPL for all applicable manufactured curb inlet protection practices. Selecting the optimal type is dependent on various site characteristics. Further, the designer must consider the level of ponding behind the device and where/how the bypass will be routed. These planning considerations ensure the device will not create public safety hazards or property damage by ponding excessive quantities of runoff or reducing sewer capacity.

Curb Inlet Protection Type 1: Block and gravel curb inlet protection devices are designed using standard concrete masonry units combined with coarse aggregate to create a temporary barrier that reduces stormwater velocities and promotes sediment settling before water enters the storm drain system. The blocks are placed lengthwise on their sides with the hollow cores oriented horizontally around the inlet opening, allowing flow to pass through while maintaining structural stability. Wire mesh (19-gauge galvanized hardware cloth), with 0.25-inch openings, is attached to the exterior faces of the blocks to prevent aggregate loss. Coarse aggregate (#57 stone) is then placed against the blocks to the top of the barrier, providing additional filtration for coarse-grained sediments.


Source: IECA (2001)

The configuration is designed to create ponding in front of the inlet, which encourages sediment deposition while still allowing overtopping during high flows. Because the height of a typical concrete masonry unit (approximately eight inches) exceeds the average curb height, overtopping is expected; however, once water flows over the curb, the large opening across the inlet face prevents excessive ponding above the curb level. Stability is often reinforced by inserting a wooden stud through the outer holes of the end blocks to secure the front row in place. While effective at reducing sediment transport, the system is prone to clogging, which significantly reduces flow-through capacity. For this reason, periodic maintenance is required, including pulling back and cleaning or replacing the gravel when it becomes clogged, which may be more intensive than maintenance regimes for other types or manufactured devices.

Curb Inlet Protection Type 2: Sandbags or bags filled with #57 stone can be stacked in an interlocking fashion to form a small dam that provides temporary curb inlet protection. For inlets located on a grade, the dam is constructed just upstream of the grate in a curved, J-hook-shaped alignment extending approximately three feet from the curb. A single bag is intentionally omitted from the upper row to create a spillway. At sag locations, the dam encircles the inlet, with one bag omitted on each side to form dual spillways. To minimize direct discharge of sediment-laden runoff, spillways should not be aligned immediately adjacent to the curb. Where feasible, the spillway should be set about one inch below curb height to reduce the potential for overtopping.



The sandbags or gravel-filled bags must be tightly packed. Where this is not achievable, geotextile fabric can be placed on the upstream face of the dam and reinforced with additional coarse aggregate (#57 stone), particularly in areas with high silt or clay content. This measure functions by slowing upstream flow velocities to promote sediment deposition, though it provides limited effectiveness during larger rainfall events due to reduced ponding depths compared to other inlet protection types. It should also be noted gravel gravel-filled bags are preferred over sandbags since sandbags themselves offer no significant filtration capacity and can easily enter the storm drain if the bag tears. Curb Inlet Protection Type 2 functions primarily by slowing upstream flow velocities so that sediments can drop out. However, the Type 2 device may create ponding depths which are somewhat lower than Type 1 or Type 3 inlet protection, and so, may not be as effective for this purpose in larger rainfall events (TDOT).

Curb Inlet Protection Type 3: Coarse aggregate wedge inlet protection consists of a wooden 2 x 4 frame placed over the curb iron, fitted with wire mesh (19 gauge galvanized hardware cloth), with 0.25-inch openings that extend over the grate, and is covered with #57 stone. This configuration reduces flow velocities within the curb, promoting sediment deposition, while the coarse aggregate provides limited filtration for larger particles. Curb Inlet Protection Type 3 can be used at both sags and on-grade inlets. Its compact design is an advantage; however, its overflow capacity is restricted to a one-and-a-half-inch slot (width of a 2 x 4), meaning flow is not conveyed until curb overtopping occurs. In sag locations, this can result in ponding several inches above the curb, while on-grade applications may experience significant bypass flow. Such a bypass can dislodge the aggregate, making this measure unsuitable for curbs with longitudinal slopes greater than one percent. In addition, locations with high sediment loads should be avoided, as the aggregate is prone to clogging (TDOT).


Source: OEPA (2006)

Curb Inlet Protection Type 4: Filter fabric inlet protection is installed by wrapping a continuous piece of geotextile fabric around the inlet grate prior to placement. At the back of the grate, the fabric is secured to a pressure-treated 2 x 4 with wire staples, which is then positioned along the gutter line above the grate. This configuration filters runoff as it enters the inlet. Type 4 devices can be used at both sag and on-grade inlets. Because they do not significantly obstruct gutter flow, ponding is unlikely at on-grade inlets; however, flow through the fabric is limited and further reduced once sediment accumulates, leading to substantial bypass, particularly where gutter slopes and velocities are high. Designers should confirm that bypassed flows can be adequately conveyed within the gutter for the design storm. At sag

locations, the 2 x 4 obstructs much of the curb opening, resulting in limited overflow capacity and potentially causing ponding depths that exceed the curb height.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

IECA. (2001). *International Erosion Control Association Photo Gallery*.

MPCA. (2023). *Minnesota Stormwater Manual: Sediment control practices – Storm drain inlet protection.*

OPEA. (2006). *Rainwater and Land Development: Sediment Controls – Storm Drain Inlet Protection.*

TDOT. *Drainage Manual Ch10*.

USEPA. (2021). *Stormwater Best Management Practice: Storm Drain Inlet Protection.*

4.4.6 Inlet Protection Filter Rings for Culverts





Source: TDOT

Definition and Purpose

Culvert inlet protection is a temporary sediment control measure installed at the entrance of culverts to reduce sediment transport into storm drainage systems before a site is stabilized. These practices are typically rock or conventional riprap structures, often combined with geotextile fabric and aggregate, which slow incoming flows, promote sediment deposition, and provide filtration. Configurations may include complete enclosures or horseshoe-shaped barriers, depending on whether the intent is to treat all inflow or isolate onsite runoff from off-site (unpolluted) run-on. By reducing flow velocity, enhancing settling, and filtering sediment, culvert inlet protection helps maintain culvert function during construction and prevents sediment accumulation and downstream transport until permanent controls and stabilization are established.

Appropriate Applications

Culvert inlet protection is appropriate in situations where culverts must be made operational before the contributing drainage area has been permanently stabilized, serving as a temporary measure to reduce sediment transport into storm drainage systems. Type 1 protection (full enclosure system) is generally applied at inlets receiving flows generated primarily from the project site and includes a sediment storage zone, while Type 2 protection (horse-shoe shaped system) is typically installed at inlets also receiving significant offsite run-on, under the assumption that runoff from undisturbed offsite areas is relatively sediment-free. The systems should be used in conjunction with other sediment controls such as silt

fences, catch basin filters, or slope drains. Type 2 systems are typically installed at larger structures, including box culverts.

Limitations and Maintenance

Culvert inlet protection has specific design limitations and requires regular maintenance to function effectively. Type 1 protection is limited to culvert inlets with contributing drainage areas up to 20 (ETWs or waters with unavailable parameters) or 30 (waters with available parameters) acres and pipe sizes of 36 inches or smaller, but it should not be installed in streams, ditches, swales, or depressions greater than one foot deep (TDOT). In cases where the drainage area exceeds the maximum acreage, the designer should implement additional EPSC measures to redirect, reduce, or detain runoff within the drainage area rather than dismiss the use of culvert protection. Drainage area limitations for Type 2 systems are determined by the materials used for constructing the measure. If a rock berm is used, the drainage area should be no more than half an acre per 100 linear feet of berm (TDOT). Allowable drainage areas for culvert protection Type 2, when constructed from other materials such as silt fence and silt fence variants (Section 4.4.10), straw wattles, and filter socks (Section 4.4.9), are presented in the appropriate sections of this manual.

All culvert inlet protection measures require frequent inspection to identify damage, clogging, or sediment accumulation. Sediment must be removed once it reaches half the design depth, with contaminated aggregate replaced as needed, and all materials disposed of in stable locations. Structures should be repaired promptly if displaced or clogged, monitored for subsidence in karst areas, and removed once upslope areas are permanently stabilized.

Planning and Design Considerations

While there is not a qualified products list (QPL) for culvert protection in Tennessee, the SedJacket is a manufactured device that can replace a constructed practice for culvert inlet protection. Before installing manufactured or constructed culvert protection, inspect the culvert inlets to identify any subsurface voids, conduits, or openings in the bedrock. Culverts operating during land disturbing activities can convey large quantities of sediment, which can lead to clogging, reduced capacity, and flooding. Ensure culvert protection designs allow for cleanout and disposal of trapped sediment, minimize interference with construction operations and vehicle traffic, and ensure that any ponded stormwater will not result in damage to nearby properties (VDEQ, 2024).

Culvert Protection Type 1 is intended for sites with drainage areas up to 30 acres, though this limit is reduced to 20 acres when discharging to ETWs or streams with unavailable parameters for siltation. The rock structure is built on a foundation of geotextile fabric,

placed in two layers beneath the mineral aggregate and conventional riprap on the downstream side of a sediment storage zone (Figure 4.4.6-A). This design prevents piping of channel bed material, reduces the risk of undermining, and enhances filtration capacity when the structure is not overtopped. Class A-1 conventional riprap is typically specified for Type 1 protection; however, Class A-3 may be used where higher sediment loads are expected, and additional filtration is required. This smaller stone size provides improved sediment capture but less structural stability, and its use is restricted to pipes up to 24 inches in diameter with drainage areas of three acres or less, or pipes up to 36 inches with drainage areas of six acres or less. The rock structure is to be trapezoidal in shape, with a top width of at least three feet, front facing slope at 3H:1V or shallower, back facing slope of 1.5H:1V or shallower, and in which the rock is at least four feet from the culvert inlet. To coincide with the rock structure, the sediment storage zone should be excavated at least one foot beneath the elevation of the rock structure and provide at least two and a half feet of sediment storage (TDOT). Unlike Type 2 systems, Type 1 systems do not require a designated overflow weir, as the entire rock assembly conveys overtopping flows (TDOT).

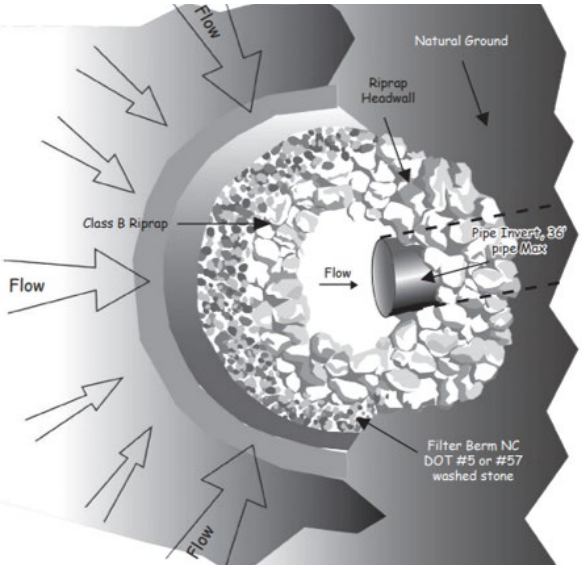


Figure 4.4.6-A: Rock pipe inlet protection plan view. Source: NCDEQ (2013).

Culvert Protection Type 2 is generally applied by wrapping a sediment control measure around the culvert endwall and extending it to the limits of the disturbed area, ensuring overlap with perimeter controls by at least two feet. Where roadway side ditches are present, the protection must connect to an enhanced rock check dam (Section 4.3.3) or a sediment trap (Section 4.4.8), and in some cases extend across the ditch to provide continuity of controls. Mineral aggregate (typically size 57) should not be used alone in ditches. Type 2 designs allow for a variety of alternate sediment control materials, including silt fence, silt fence variants,

filter socks, or sediment tubes (TDOT; VDEQ, 2024). These measures are arranged in a horseshoe configuration around the inlet and tied into adjoining measures to prevent bypass flow (Figure 4.4.6-B). For example, silt fence inlet protection should be installed approximately six feet in front of the culvert, with a height between 16 and 34 inches, a maximum drainage area of one acre, and an expected service life of up to three months (VDEQ, 2024). Filter socks or straw wattles may substitute for silt fence when staked to the required minimum height. For larger contributing drainage areas, culvert inlet sediment traps may be used (Figure 4.4.6-C), with drainage limited to three acres, a maximum berm height of three feet, side slopes of 2H:1V for stone berms and 1H:1V for excavated storage, and a wet storage depth of no more than four feet. These traps combine coarse aggregate and Class A-1 conventional riprap for effective sediment filtration, with filter fabric separating stone from soil to prevent undermining. Together, these specifications ensure that Culvert Protection Type 2 can be tailored to site conditions, offering flexibility in material selection while maintaining reliable sediment capture and conveyance capacity.

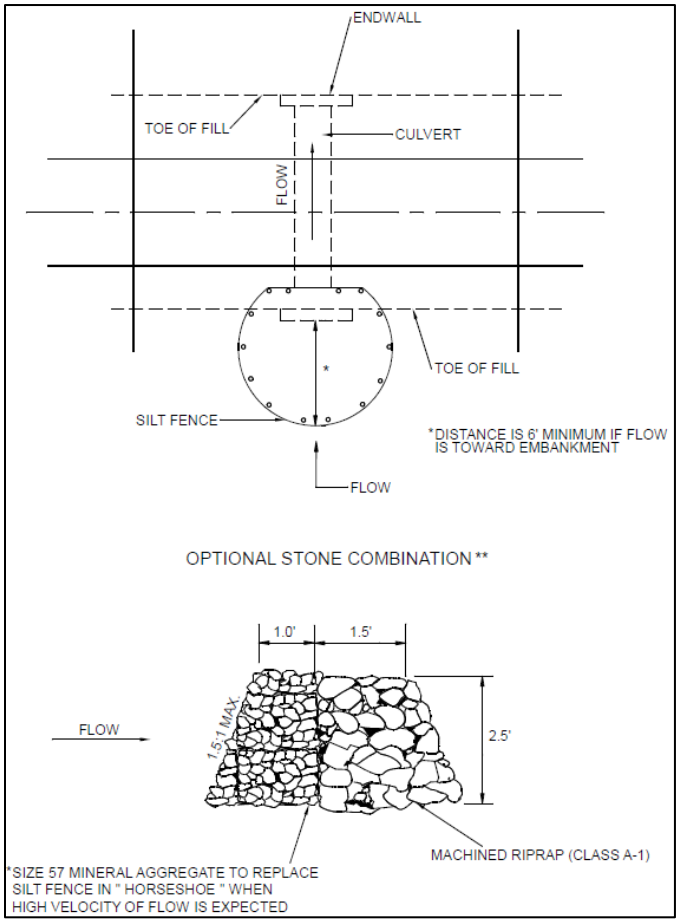


Figure 4.4.6-B: Silt fence culvert inlet protection details. Adapted from VDEQ (2024).

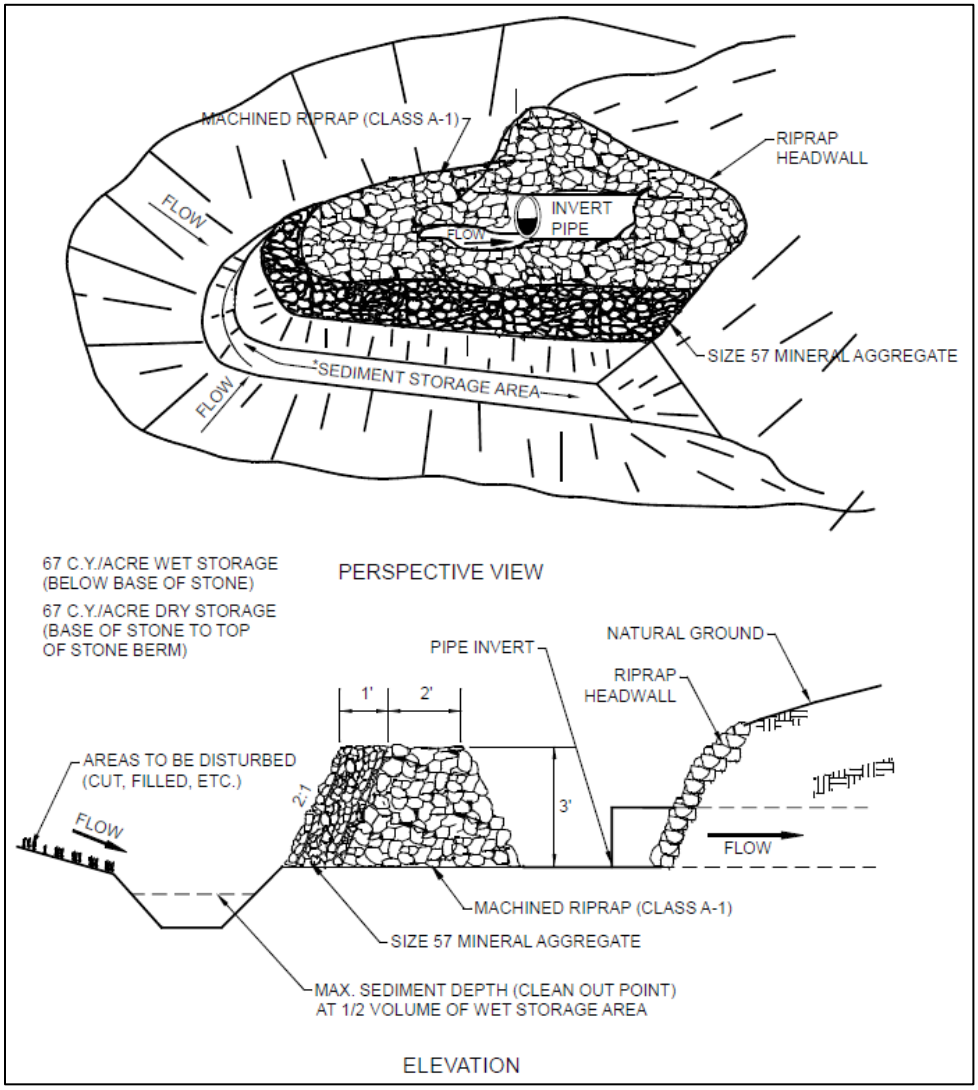


Figure 4.4.6-C: Culvert inlet sediment trap details. Source: Adapted from VDEQ (2024).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.
TDOT. *Drainage Manual Ch10*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.4.7 Temporary Sediment Basin



Source: City of Franklin

Definition and Purpose

A sediment basin is a temporary impoundment structure, typically constructed by excavating a depression or building a compacted embankment, designed to intercept and detain sediment-laden stormwater runoff from active construction sites. The primary components of a sediment basin include a forebay, an earthen embankment, sediment storage, live storage, surface dewatering mechanism, principal spillway, and emergency spillway. These basins slow the velocity of stormwater, allowing suspended particles time to settle out before being discharged offsite. Beyond sediment control, these basins can function as temporary peak flow attenuation systems to mitigate downstream erosion. While intended to be temporary measures, sediment basins can be converted into permanent stormwater controls with proper planning and design (Section 7).

Appropriate Applications

Sediment basins, or their engineered equivalents (Section 5), are to be used where treatment of sediment-laden runoff is necessary, as specified by the Tennessee CGP. They are most appropriately located at the lowest parts of the site, typically along the perimeter of an area to be disturbed, where flows concentrate before discharging offsite. For drainage areas less than (approximately) five to 10 acres, other sediment control measures are likely more economical and effective. Typically, sediment basins are used when the contributing drainage area is less than 100 acres. (USEPA, 2021).

Limitations and Maintenance

Sediment basins may be difficult to construct or less effective when a construction site has steep slopes, shallow bedrock, or other site constraints. Site conditions may determine that

alternatives to sediment basins are better suited and more effective. Further, multiple, less extensive measures can be implemented in series to obtain the same, or greater, sediment removal capacity (Section 5) than a sediment basin. Equivalent control justification should be provided in the EPSC plans when applying for coverage under the CGP. See Chapter 5 for additional details on equivalent measures.

When inspecting sediment basins, assess the structural integrity, sediment accumulation, and the condition of spillways, embankments, and outlet structures. When 50% of the sediment storage capacity of the sediment basin is filled, accumulated sediment must be removed and disposed of properly or stabilized onsite. Such maintenance requires equipment access to the basin and dewatering operations. To ease sediment removal operations, forebays are used as a form of pretreatment that captures larger sediment particles and reduces maintenance to a localized area. Forebays also require sediment removal maintenance once 50% of the sediment storage capacity has been reached; however, they are designed to completely dewater through gravity, making sediment removal operations more efficient compared to the main basin. Routine maintenance tasks for sediment basins, in addition to sediment removal, include removing debris from risers, dewatering devices, and spillways, repairing erosion or piping along the embankment and outlet pipe, inspecting for signs of structural failure, such as subsidence, cracking, or bulging, and visual observation of the quality of water discharging from the basin during and/or after storm events.

Lastly, sediment basins are subjected to the regulations of the Tennessee Safe Dam Act of 1973 if either the embankment is 20 feet or more in height (measured from its crest down to the lowest point of natural grade, which is the downstream toe of the embankment) or if the impoundment will have a capacity, at maximum water storage elevation, of 30 acre-feet (48,400 cubic yards) or more. When either condition is met, refer to Section 1.3.2 for appropriate permitting. Limiting sediment basin drainage areas to less than 25 acres and diverting unpolluted runoff around the construction site are ideal means to lessen the likelihood of meeting the criteria of the Tennessee Safe Dam Act.

Planning and Design Considerations

Planning Considerations

Sediment basins need to be designed with proper planning to ensure effectiveness, regulatory compliance, and economic installation. First, the location of a sediment basin is critical to its effectiveness. Not only does its location impact sediment removal capabilities, but sediment basins must be located such that sudden failures do not result in loss of life or

serious property damage. For maximum effectiveness, sediment basins should be located to intercept the largest possible amount of sediment-laden runoff from the area that is to be disturbed (ALSWCC, 2018; TDOT). Further, sediment basin locations impact the basin design dimensions, orientation, and accessibility. Basins are not to be located in wetlands, streams, buffer zones, or on steep, unstable slopes, as these conditions compromise effectiveness and increase risk of failure (VDEQ, 2024). Second, sediment basins are commonly converted to permanent (i.e., post-construction) stormwater control measures such as detention basins (Section 7.3) and retention basins (Section 7.4), because many of the design principles complement one another. In such cases, to ensure regulatory compliance, both construction *and* post-construction regulations (local and state) are to be incorporated into the design. When the sediment basin is not to be converted to a permanent stormwater control, the embankment and resulting sediment deposits are to be leveled (and stabilized) or otherwise disposed of properly. Third, to ensure an economic and effective design, diverting runoff (Section 4.2.2) from undisturbed areas reduces the volume of runoff a sediment basin must treat. Furthermore, when a sediment basin is to be converted to a permanent post-construction stormwater control measure, consider a design that will overlap the design components and earthwork, such that they meet both construction and post-construction regulations. Because many of the design considerations overlap, construction of a separate, permanent basin is likely unnecessary and can be achieved through a simple conversion of the temporary sediment basin.

All state and local health and safety requirements shall be met concerning fencing and signs warning the public of the hazards of soft, saturated sediment and flood water. The designer and developer should be aware of the potential hazards that a temporary sediment basin represents to the health and safety of a neighborhood. Sediment basins can be attractive to children and can be dangerous to those who may accidentally slip into the water and soft mud or who may become entrapped at flowing inlets. The basin area should, therefore, be fenced or otherwise made inaccessible, unless this is deemed unnecessary due to the remoteness of the site. Strategically placed signs around the impoundment should also be installed.

Special consideration may need to be given in design, operation and maintenance in areas of the state where health hazards stemming from mosquito breeding and West Nile Virus have occurred. In any case, local ordinances and regulations regarding health and safety must be followed. Setting the sediment storage depth (i.e., permanent pool) greater than four feet helps limit mosquito breeding (Zawarus, 2022). Further, ensuring the criteria of live storage dewatering between 48 and 72 hours also helps prevent mosquito breeding (Henn et al., 2008; Zawarus, 2022).

Sediment basins are subjected to the regulations of the Tennessee Safe Dam Act of 1973 if either the embankment is 20 feet or more in height (measured from its crest down to the lowest point of natural grade, which is the downstream toe of the embankment) or if the impoundment will have a capacity, at maximum water storage elevation, of 30 acre-feet (48,400 cubic yards) or more. When either condition is met, refer to Section 1.3.2 for appropriate permitting. Limiting sediment basin drainage areas to less than 25 acres and diverting runoff associated with undisturbed areas around the construction site are ideal means to lessen the likelihood of meeting the criteria of the Tennessee Safe Dam Act.

Design Considerations

Sediment basins have various design components and are to be installed before any upgradient earth-disturbing activities commence. Sediment basin design components include a forebay, an earthen embankment, sediment storage, live storage, a surface dewatering mechanism, a principal spillway system, and an emergency spillway (Figure 4.4.7.1-A). The following are shortened definitions and functions of each component:

- *Forebay* – pretreatment mechanism meant to settle the majority of coarse sediments, thereby reducing the sediment removal frequency of the basin. The forebay is most commonly attached to the basin; however, it can be detached and positioned upgradient of the basin;
- *Embankment* – an earthen structure, either excavated or built and compacted. It functions to detain runoff;
- *Sediment storage* - bottom layer within the primary basin footprint. It is often referred to as wet storage, dead storage, or permanent pool, and is where soil particles settle and accumulate. A dewatering mechanism sets the height below which water is pooled, unless dry weather conditions or infiltration lower the water level;
- *Live storage* – a zone in between the sediment storage and top of principal spillway that is sometimes referred to as dry storage. The live storage zone is sized hydrologically to capture and slowly release runoff from the design storm;
- *Dewatering mechanism* – a mechanism, such as a perforated riser (Section 4.4.12.2.1) or a skimmer (Section 4.4.12.2.2), that slowly releases water from the live storage zone. The controlled outflow of water allows time for soil particles to settle out and reduces peak flows. This mechanism sets the top elevation of the sediment storage zone and the beginning elevation of the live storage zone;
- *Principal spillway* – a component in which the dewatering mechanism is either built into or attached to. The spillway conveys water from the basin to the downstream side of the embankment through a pipe (barrel);

- *Emergency spillway* – a bypass spillway designed to safely convey the 25-year, 24-hour storm event. It provides a safe pathway for larger storms to bypass the embankment and principal spillway, thus ensuring resiliency of the basin. Some local jurisdictions may require the emergency spillway to convey a peak flow rate associated with a larger design storm.

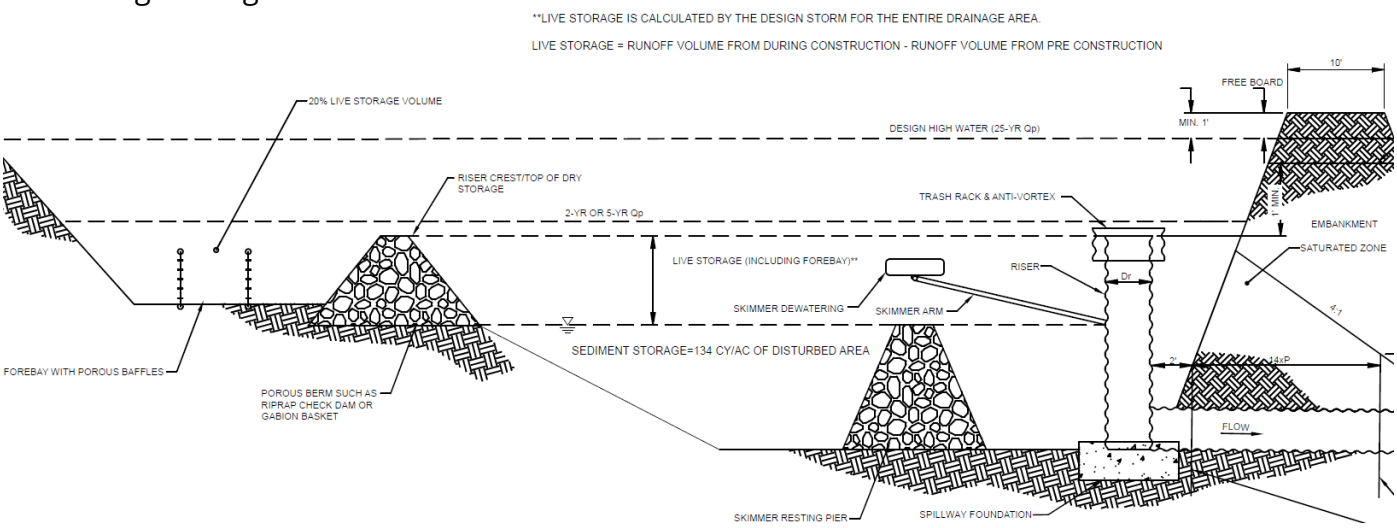


Figure 4.4.7.1-A: Sediment basin components.

Live Storage Design

Volume is one of the most important aspects of sediment basin design. Live storage is the area above the sediment storage that fills with runoff during a storm event and is therefore sized from the site hydrology based on the design storm. At a minimum, live storage volume should have a capacity of the difference in runoff volume between construction conditions (typically assumed as bare soil for the disturbed area and meadow for the areas left undisturbed) and pre-construction conditions (typically assumed as a meadow). Ensure land use (or multi-land use) assumptions accurately reflect site conditions.

The live storage volume will drain through the dewatering mechanism. In its volumetric design, live storage should have a minimum length (L) to width (W) ratio of 2L:1W, which, within reasonable bounds, increases detention time, trapping efficiency, and reduces short-circuiting (Haan et al., 1994). At higher velocities of inflow, short-circuiting or dead space can become an issue. To prevent inflow from this phenomenon, such that influent sediments disperse over the entire surface area, porous baffles (Section 4.4.12.1.1) can be implemented into sediment basin design. Length is defined as the distance between a point of inflow and the principal spillway. Where space limitations do not accommodate the minimum L:W ratio, solid baffles (Section 4.4.12.1.2) can be used to increase flow path lengths. Where there is

more than one inflow point and where circumstances preclude this ideal arrangement, any inflow point that conveys more than 30% of the total peak inflow rate should meet the minimum L:W ratio. Embankment side slopes, known as the horizontal (H) to vertical (V) ratio, should not be steeper than 2H:1V for stability. The combination of required live storage volume, L:W ratio, and H:V ratio will help determine the dimensions of the live storage zone and its surface area at capacity. Sediment trapping efficiency is primarily a function of particle size distribution, and thus the design of sediment basins should account for the sizes of influent particulates (Haan et al., 1996). However, such design procedures require complex analyses. To compensate for this, a larger surface area, particularly in terms of surface area to inflow, has been found to promote the settling of particulates. Barfield & Clar (1986) suggest that live storage surface area at capacity (acres) should be at least 100 times the peak inflow (cubic feet per second). Thus, the following equation is a good design check:

$$A_s \geq 0.01 \times q_p \qquad \text{(Eqn 28)}$$

where A_s is the surface area of the live storage at capacity and q_p is the peak flow rate corresponding to either the 2-year, 24-hour or 5-year, 24-hour storm event and calculated from general engineering methodologies as specified in Section 2.1.3. Sediment trapping efficiencies in the live storage zone can also be improved by incorporating flocculants and polymers (Section 4.4.12.5); however, these technologies should be implemented in pretreatment or upstream of the sediment basin in order for them to take effect and assist in sediment settling in the basin.

Sediment Storage Design

While live storage is calculated hydrologically, sediment storage volume is calculated based on the anticipated sediment yield. To reiterate, the sediment storage zone is often referred to as wet storage, dead storage, or the permanent pool zone in other manuals or design guidance documents; herein, it will be referred to as sediment storage. Minimum sediment storage volume can be calculated via the sediment yield ratio. The sediment yield ratio assumes one inch of erosion and sediment yield over one acre per year, which equates to 3,618 cubic feet per acre per year of sediment. The sediment storage volume should be calculated from the disturbed area within the contributing drainage area. However, site specific calculations should also be checked, especially for larger sites. If these calculations yield greater volumes than the sediment yield ratio, site specific calculations must be followed. If these calculations yield lesser volumes, the sediment yield ratio can be used. Site specific calculations can be evaluated with methods expressed in Section 2.2. It may be beneficial to add physical elevation or height markers within the basin footprint or on the principal spillway to assist in assessing accumulated sediment height in the sediment storage zone. Once the sediment has accumulated to 50% of the sediment storage volume or within one foot of the dewatering mechanism, sediment removal is necessary. The sediment

storage volume will have side slopes, which are reflected in H:V ratios, with recommended ratios of 2H:1V or flatter. Since there is a slope on the basin sides, 50% storage capacity will not occur at half the sediment storage height. The sediment storage volume and specified L:W and H:V ratios can help determine the sediment storage height, which will be used to set the elevation of the dewatering mechanism. Overall, the height and volume of sediment storage promote better settling conditions between runoff-producing events and minimize re-suspension of captured sediment. Relatively little research exists specifying a minimum sediment storage depth that substantially reduces or minimizes sediment resuspension due to influent velocities and wind, though it is known that a larger depth will continue to decrease resuspension. IECA (2021), MPCA (2023), and ALSWCC (2022) recommend minimum sediment storage depths of one, one and a half, and two feet, respectively. A Pennsylvania-based study found that a sediment basin with a sediment storage depth of one and a half feet discharged 23.1 kilograms of sediment from two two-year storms. Of the 23.1 kilograms, approximately 13% of the sediment was resuspended and discharged from previously settled sediments (Fennessey & Jarrett, 1997). A sediment storage depth of four feet (or greater) discourages mosquito breeding (Zawarus, 2022) but is rather impractical for construction purposes. A sediment storage depth of four feet would likely yield a vast overdesign in most scenarios. Lastly, if a sediment basin is to be converted to a permanent detention or retention basin, consider over-excavating the sediment storage such that accumulated sediment will bring the bottom of the basin to permanent basin grade. With this method, accumulated sediment will not need to be removed from the basin before conversion to a permanent stormwater control; however, it will require stabilization.

Forebay Design

The forebay is most effective when positioned at the primary inlet of the sediment basin, where it serves to intercept incoming flow and facilitate the settling of larger particles prior to entering the main basin. To maximize performance, the forebay should comprise at least 20% of the live storage volume. Its bottom elevation should match the top of sediment storage elevation in the primary basin, such that the forebay does not have a permanent pool, and should be separated from the main basin by a porous barrier, such as a rock berm, check dam, etc. This barrier promotes sedimentation of larger particles and diffuses incoming flow, reducing the risk of short-circuiting. The crest of the berm must not exceed the elevation of the principal spillway riser crest and be designed to convey



the 2-year or 5-year, 24-hour peak flow rate via a designated spillway. To prevent re-suspension of accumulated sediment and minimize scour during high flow events, influent energy must be controlled through the use of an energy dissipator such as plunge pools, conventional riprap, or other appropriate measures (Section 4.3.4). Additionally, the forebay must be easily accessible for routine maintenance, as it will accumulate sediment at a faster rate than the rest of the sediment basin. Accumulated sediment must be removed and disposed of properly once 50% of the forebay capacity has been reached.

The forebay may also be completely disconnected from the main basin and in the influent channel. Emmett (2022) compared sediment basin performance between three design configurations: 1) 2012 TDEC recommendations (which included a constructed 20% live storage forebay volume, 2) TDOT recommendations without a forebay, and 3) TDOT recommendations with a rock check dam placed in an influent channel. While all three designs exceeded the 80% sediment removal criteria within Tennessee, the influent channel check dam improved the performance of the standard TDOT sediment basin by removing an additional 3% total mass of sediments, making it nearly as effective as the standard 20% live storage forebay. The rock check dam acting as a forebay configuration removed between 63 and 75 percent of the total sediments (before entering the main basin), which was similar to the 68 to 80% of the 2012 TDEC forebay, demonstrating the impact of forebays in reducing sediment removal maintenance from the main basin. Additionally, the rock check dam forebay was credited with further reducing effluent peak flow rates and achieved the 80% sediment removal criteria quicker than when it was not implemented. In all regards of the study, the rock check dam forebay performed nearly as well as a typical forebay design, while decreasing the forebay footprint (Emmett, 2022).

Principal Spillway Design

The principal spillway consists of a vertical riser, typically constructed from corrugated metal or reinforced concrete, connected to a horizontal drainpipe (barrel) that extends through the embankment and discharges beyond the downstream toe. The recommended minimum temporary riser diameter for a circular riser pipe is 18 inches or equivalent cross-sectional area for other shapes. The minimum dimensions are recommended for ease of maintenance. Further, a trash rack and anti-vortex device may be necessary to reduce clogging in the riser. The foundation base of the principal spillway should be firmly anchored to prevent floating due to buoyancy. Computations, with a minimum factor of safety of 1.25 (downward forces = 1.25 × upward forces), to determine anchoring specifications should be completed. For risers 10 feet or less in height, the anchoring may be a concrete base or a square steel plate. The concrete base needs to be at least 18 inches thick, with the edge of

the concrete base extending past the riser a distance equal to (or greater than) the diameter divided by two (Figure 4.4.7-B). When using a rectangular riser structure, replace diameter with the width of the box. Also, the riser should be embedded into the concrete at least six inches for stability. The square steel plate, which is used for a metal riser, should have a minimum thickness of one-fourth inch, a width equal to twice the diameter of the riser, and be welded to the riser pipe. The plate is to be covered in at least two and a half feet of stone, gravel, or compacted soil to counteract buoyancy. If compacted soil is the chosen method, it should be compacted to 95% maximum proctor density. If a steel plate is employed, ensure that material over the plate is not removed during maintenance activities.


Source: City of Franklin, TN

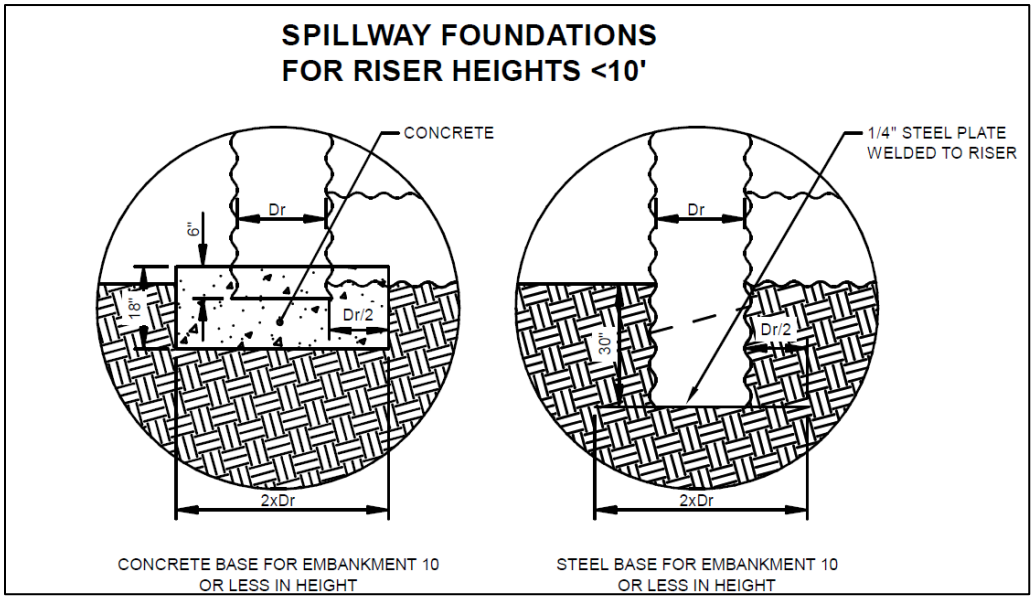


Figure 4.4.7-B: Riser pipe details.

Sizing of the barrel, which extends through the embankment to an outlet, can be computed utilizing Eqn 15 (a reformulated version of Manning's Equation) and methodology from Section 3.3.2.4. However, at a minimum, the barrel should be 12 inches in diameter to minimize clogging and allow maintenance. The riser and all pipe connections shall be mechanically sound and completely watertight and not have any other holes, leaks, gashes, or perforations other than designed openings for dewatering. Do not use dimple (mechanical) connectors as they are not watertight. The use of plastic pipe through the

embankment should be approached with caution due to the potential for deflection, long-term creep, and separation from the surrounding soil. The outlet of the barrel must be adequately protected or fitted with energy dissipation measures [e.g., riprap aprons (Section 4.3.6), stilling basin (Section 4.3.7), baffles (Section 4.3.4), etc.] to prevent erosion. Where discharge occurs at or near property boundaries, appropriate drainage easements should be obtained in compliance with local ordinances. Additionally, it is best when outlet flows from the impoundment are directed into a natural receiving watercourse to preserve existing flow paths and avoid adverse impacts to downstream or adjacent offsite properties.

In order to prevent internal erosion in the embankment, the fill must be carefully compacted around the drainpipe barrel. If good compaction methods are difficult to implement, anti-seep collars can be used to form structurally sound, watertight connections. Anti-seep collars help reduce uncontrolled seepage and prevent internal erosion or piping inside the embankment and along the drainpipe barrel. Unfortunately, poor compaction and construction techniques are common, and therefore some localities require anti-seep collars instead of allowing the alternative of quality compaction around the drainpipe. When used, they are to be placed on the drainpipe barrel of the principal spillway within the normal saturation zone of the embankment to increase the seepage length by at least 10%, under either of the following conditions:

- The settled height of the embankment exceeds 10 feet; or
- The embankment has a low silt-clay content (Unified Soil Classes SM or GM), and the barrel is greater than 10 inches in diameter.

Since the anti-seep collars are to be installed within the saturated zone, the assumed normal saturation zone is determined by projecting a 4H:1V sloped line from the point where the normal water elevation (can be assumed to be the top of the principal spillway) meets the upstream slope to a point where this line intersects the invert of the barrel pipe or bottom of the cradle, whichever is lower. The collars should extend a minimum of two feet around the barrel. The maximum spacing between collars should be 14 times the projection of the collars above the barrel and not closer than two feet to a pipe joint. Collars should be placed sufficiently far apart to allow space for hauling and compacting equipment. Precautions should be taken to ensure that 95% compaction is achieved around the collars. Connections between the collars and the barrel shall be watertight. Plans should specify the method of compaction around the pipe barrel to ensure adequacy and to prevent damage to the anti-seep collars and joints. Follow compaction specifications provided by the project engineer to ensure adequacy and to prevent damage to the anti-seep collars and joints.

Dewatering Mechanism

A dewatering mechanism is typically either a perforated riser pipe or a skimmer attached to the principal spillway. Dewatering openings should be sized to drain the live storage volume within 48 to 72 hours, thereby allowing ample time for settling of soil particles but draining before another storm. Furthermore, as previously mentioned, dewatering within this timeframe (i.e., a constantly changing water surface elevation) discourages mosquito breeding (Henn et al., 2008; Zawarus, 2022). Design guidance for dewatering mechanisms can be found in Section 4.4.12.2. Pumps can also be a means to dewater a basin. When using a portable pump, the inlet hose connected to the pump should be secured so that it only draws down water from the basin surface and cannot sink below the surface. Adequate energy dissipators are also to be in place at the outlet end of the discharge hose to prevent scouring and erosion. Treatment when pumping can be placed at the inlet or outlet of the system, such as a skimmer (Section 4.4.12.2.2) or a sediment filter bag (Section 4.4.12.6).

Emergency Spillway

The emergency spillway serves as a critical safety mechanism for a sediment basin by conveying flows from storm events exceeding the design storm. The spillway should be sized to pass the peak flow from the 25-year, 24-hour storm, without overtopping or compromising the integrity of the embankment. Thus, no flow should leave the basin through the emergency spillway during the design (2-year or 5-year, 24-hour) storm. It also functions as an emergency outlet in the event of excessive sediment accumulation, reducing the storage of the basin, blockage of the principal spillway, or failure of the dewatering mechanism, which would prevent proper dewatering times. An evaluation of the site and downstream conditions must be made to determine the location of an emergency spillway. In some cases, the site topography does not allow a spillway to be constructed in undisturbed material. A minimum of one foot of vertical height is needed between the top of the principal spillway and the emergency spillway elevation. There should also be one foot of freeboard between the 25-year, 24-hour high-water elevation and the embankment crest elevation. Emergency spillways should be constructed on undisturbed ground or well-consolidated soil in order to reduce the risk of settlement,


Source: City of Franklin

decreased freeboard, and scouring during the design storm. Emergency spillways should be lined with non-erodible material appropriate to withstand the calculated shear stress within the channel from the 25-year, 24-hour storm event and over-excavated to compensate for the thickness of linings such as conventional riprap in order to preserve its intended design flow capacity. A professional engineer is responsible for the design of the emergency spillway.

Embankment

Embankments are earthen structures that are built up and compacted. It provides the total storage, which is comprised of the sediment storage zone, live storage zone, and all additional freeboard requirements. In addition to the required one foot of vertical elevation between the top of the principal spillway and the bottom of the emergency spillway, one foot of freeboard should be provided from the 25-year, 24-hour highwater elevation and the embankment crest. The top width of the embankment is dependent on embankment height (Table 4.4.7.1-A) and should not be less than 10 feet wide if the embankment is used for maintenance access. All components of the embankment must be stabilized upon construction completion and before it becomes operational. Trees and/or shrubs should not be allowed to grow upon the embankment due to the ability of the roots of such vegetation to destabilize the embankment and/or encourage piping.

Table 4.4.7.1-A: Embankment with specifications. Source: USDA (2022).

Embankment Height (ft)	Embankment Width (ft)
< 10	6
10 - 14	8
15 - 19	10
20 - 24	12
25 - 34	15

For earth-fill embankments, a cutoff trench should be along the embankment centerline to minimize seepage and reduce the risk of internal erosion. This trench is to extend a minimum of one foot into a stable, impervious soil layer and be at least two feet deep. It should also extend up both abutments to the elevation of the riser crest. The minimum width of the trench is four feet, or wider as necessary to accommodate compaction equipment. Side slopes should not be steeper than 2H:1V. Compaction of the trench must meet the same standards as those specified for the embankment. During backfilling and compaction, the trench should be adequately drained.

Embankment fill material should be clean mineral soil taken from borrow areas and free of roots, woody vegetation, stumps, sod, oversized stones, rocks, or other perishable or objectionable material. The fill material selected must have enough strength for the dam to remain stable and be cohesive when properly compacted, to prevent excessive percolation of water through the dam. Any embankment material should contain approximately 20% clay particles by weight. Using the Unified Soil Classification System, SC (Clayey sand), GC (clayey gravel), and CL ("low liquid limit" clay) are among the preferred types of embankment soils. Areas on which fill is to be placed shall be scarified prior to placement of fill. The fill material should contain the amount of moisture needed to ensure that at least 95% compaction will be achieved, and must be placed in six-inch lifts over the entire length of the fill. Loosely placed embankment soil is subject to excessive settlement, severe erosion, and slope failure. Compaction can be obtained by routing the hauling equipment over the fill so that the entire surface of the fill is traversed by at least one wheel or tread track of the equipment, or by using a compactor. Note that the spillway barrel must be installed in the embankment as it is being constructed in lifts and proper compaction is occurring around the barrel, especially under the haunches. If used, special care shall be taken in compacting around the anti-seep collars (compact by hand, if necessary) to avoid damage and achieve the desired compaction. The embankment should be constructed to an elevation 10% higher than the design height to allow for settlement. If compaction equipment is used, the overbuild may be reduced to not less than 5%. Areas under the proposed embankment (or any structural works related to the sediment basin) shall first be cleared, grubbed, and stripped of topsoil. All trees, vegetation, roots, and/or other objectionable or inappropriate materials should be removed and disposed of by appropriate methods.

Additional Sediment Basin Enhancements

A valve to close the dewatering mechanism or principal spillway can be used to prevent the basin from discharging, so that it provides additional settling time. The longer the residence time, the more sediment will fall out of the water due to gravity. For further enhancement, flocculants and polymers (Section 4.4.12.5) can be applied to the water. Chemical application can be applied directly to the water in the sediment basin or in a channel leading to the sediment basin. If it is applied directly to the water, it needs to be properly agitated to ensure appropriate mixing. If this is incorporated, the SWPPP is to specify that the gate valve is to remain closed (i.e., holding back all the water in the live storage zone) until it has been chemically treated, the water agitated, and sufficient time has passed to allow sediment to settle (Figure 4.4.7.2-C). Typically, this is at least 24 hours or per manufacturer recommendations. The live storage should be able to dewater within a time frame of 48 to 72 hours when the valve is open.



Figure 4.4.7.-C: Visible differences of the turbidity upon initial chemical application (left) and 24 hours after chemical application (right).

Example Application

Given:

A project site discharges to a stream that has unavailable parameters due to siltation and the following site conditions:

- Drainage area of 10 acres;
- Disturbed area of 7 acres;
- Rainfall of the 5-year, 24-hour storm is 4.08 inches;
- Assume disturbed areas have a curve number of 91;
- Assume undisturbed areas have a curve number of 71;
- Time of concentration during construction of 7.4 minutes;
- Time of concentration pre-construction of 12 minutes;
- Incoming 5-year, 24-hour peak flow rate during construction of 41 cfs; and
- Incoming 25-year, 24-hour peak flow rate during construction of 58 cfs.

Determine:

The design of a sediment basin utilizes the following assumptions:

- Uniform basin geometry - trapezoidal volume and a flat rectangular bottom surface;
- Forebay storage is to be 20% of the live storage;
- Sediment yield of 3,618 ft^3/ac of disturbed area (or 134 yd^3/ac);
- Total live storage provided must capture the increased volume of runoff between pre-construction conditions and during construction conditions; and
- The volume of a trapezoid is approximated by:

$$V = \frac{(A_t + A_b) \times d}{2}$$

where V is the volume of the basin, A_t is the basin top surface area, A_b is the basin bottom surface area, and d is the incremental or total depth.

Solution:

Step 1 – Determine the required sediment storage volume: Sediment storage volume is computed using the sediment yield ratio of 3,618 ft^3/ac of disturbed land, which was based on the assumption of sediment clean out at least every six months.

$$\text{Sediment storage} = 3{,}618 \times 7 = 25{,}326 \text{ ft}^3$$

Step 2 – Determine the required live storage volume: The required live storage volume is computed by the difference in runoff volume between construction and pre-construction conditions. Using the NRCS methodology to compute runoff volume (i.e., Eqns 1-3) and a total drainage area of 10 acres, runoff volume can be computed for both scenarios as shown below (refer to Section 2.1.1 for nomenclature). The composite curve number approach (Eqn 4) was used for the construction scenario where seven acres of land were disturbed and the three acres were left undisturbed.

Scenario	CN_{comp}	S (in)	λ	I_a (in)	Q_{CN} (in)	Runoff (cf)
Pre-Construction	71	4.085	0.2	0.817	1.449	52,604
Construction	85	1.765	0.2	0.353	2.529	91,811

The required live storage volume is the difference between the two, which equates to 39,207 ft^3.

Step 3 – Determine the forebay storage volume: The forebay storage volume is assumed to be 20% of the live storage volume or 7,841.4 ft^3.

Step 4 – Determine the total basin storage volume: The total basin volume is the sum of sediment storage volume and live storage volume, or 64,533 ft^3.

Step 5 – Determine the minimum surface area at the riser elevation, A_s: The minimum surface area can be computed from Eqn 28

$$A_s \geq 0.01 \times q_p = 0.01 \times 41 = 0.41 \text{ ac} = 17{,}860 \text{ ft}^2.$$

Therefore, the surface area of live storage at the riser elevation should be 17,860 ft^2 or larger.

Step 6 – Determine the geometry of the live storage zone: Once the required volumes and surface area are known, the dimensions and depths are variable depending on site constraints (available area and depth). In order to get the required volume, the basin can have a larger surface area and be shallow or deep with less surface area. There

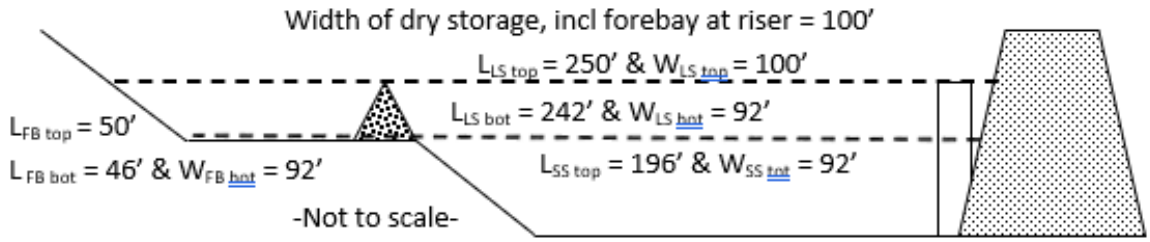
is room for the engineer to design what is best for the site. The following is an example methodology for a basin to meet the calculated volumes and best practices.

Step 6a:

The live storage geometry is a trial and error approach by first assuming a depth. Trialing a depth of 2 feet, ensure the surface area condition is met:

$$S.A = V / D = 39{,}207 / 2 = 19{,}603.5 \text{ ft}^2$$

The calculated surface area based on the assumed depth is larger than the minimum surface area of 17,860 ft^2 and therefore passes the first check. Next, assume a basin shape with a L:W ratio of 2. First, assume a length of 250 feet and a width of 100 feet at the top of live storage, which equates to a surface area of 25,000 ft^2 as shown below.



Calculate the live storage volume to see if it provides more than the required storage volume.

$$V = [(250 \times 100) + (242 \times 92)] \times 2 / 2 = 47{,}264 \text{ ft}^3$$

The surface area for the bottom of live storage was calculated by using the slope of 2H:1V. If the depth of the live storage is 2 ft, then the slope of the embankment will taper to the bottom of the live storage over 4 ft on one side. Since there are two sides, that means the bottom of live storage length is 8 ft less than the top of live storage length, which is where 242 ft originated. A similar calculation was done for the width. The forebay dimensions were approximately 20% of the live storage dimensions; however, length of length of the forebay at the bottom only decreased from one side as the forebay barrier dimensions were assumed to be negligible (i.e., the forebay did not contribute to an H:V ratio). The calculated volume is greater than the required 39,207 ft^3 of live storage, and therefore, the proposed geometries are acceptable.

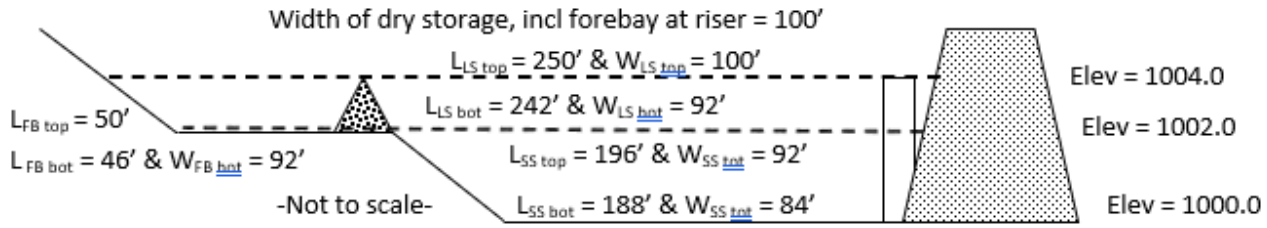
Step 6b:

If the volume calculated in step 6a was not greater than the required, the engineer can amend the depth or L:W or H:V ratios as a best fit. Ensure the minimum surface area recommendation is still met. Then repeat the calculations until the calculated volume is greater than the required. Avoid large overdesign.

Step 7 – Determine the geometry of the sediment storage zone: The sediment storage geometry is also trial and error approach by first assuming a depth. Trialing a lesser depth initially is recommended as the footprint will be minimized, and excess excavation is avoided. Trialing a depth of 2 ft with sediment storage width consistent with the rest of the sediment basin, as it continues down 2 more feet and a side slope of 2H:1V, ensure the geometry provides adequate sediment storage capacity. A depth of 2 ft at a 2H:1V slope is 4 ft on each side, so the bottom width of the sediment storage is 84 ft. If using a 2L:1W ratio, it can be calculated that the length is 234 ft. Therefore, the sediment storage volume can be calculated.

$$V= [(196 \times 92) + (188 \times 84)] \times 2 / 2 = 33{,}824 \text{ ft}^3$$

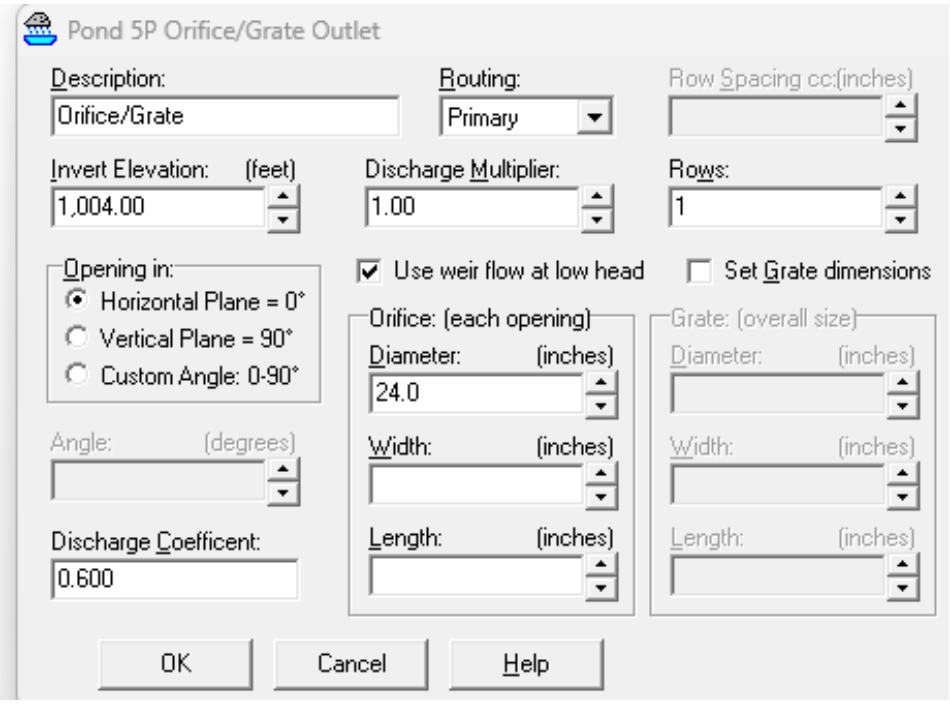
The calculated sediment storage volume is greater than 25,326 ft^3, the required volume; therefore, these dimensions are acceptable.



Step 8 – Summary: With the calculated dimensions, it can be assumed the following are acceptable:

- Riser height = 4 ft;
- Bottom basin elevation = 1000.0 ft;
- Top of sediment storage elevation = 1002.0 ft; and
- Top of riser elevation = 1004.0 ft.

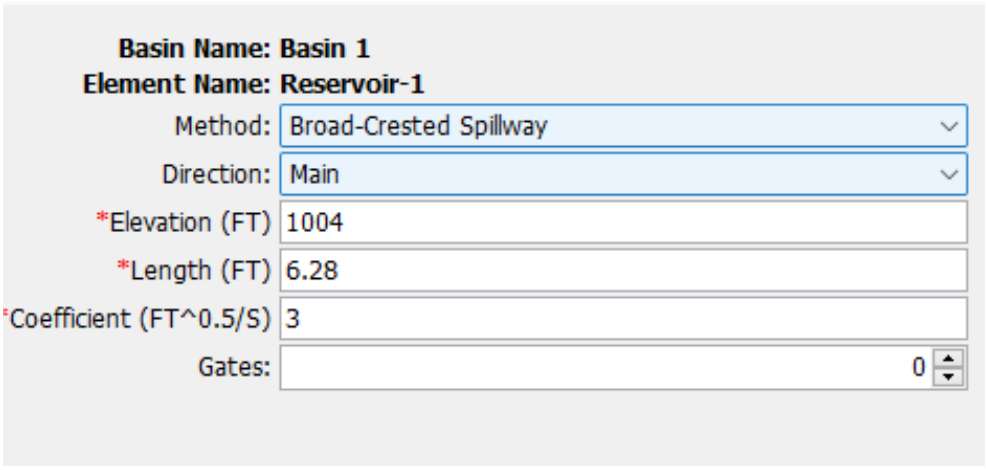
Step 9 – Design of the principal spillway: Establish the principal spillway elevation of 1004.0 ft and assume a riser diameter of 24 inches. For a standpipe in HydroCAD, select the outlet as an orifice/grate in the horizontal plane, as shown below. HydroCAD gives an elevation of 1004.12 ft for the 5-year, 24-hour storm with a 24-inch riser pipe. This is acceptable. If desired, a smaller pipe size could be modeled and selected.

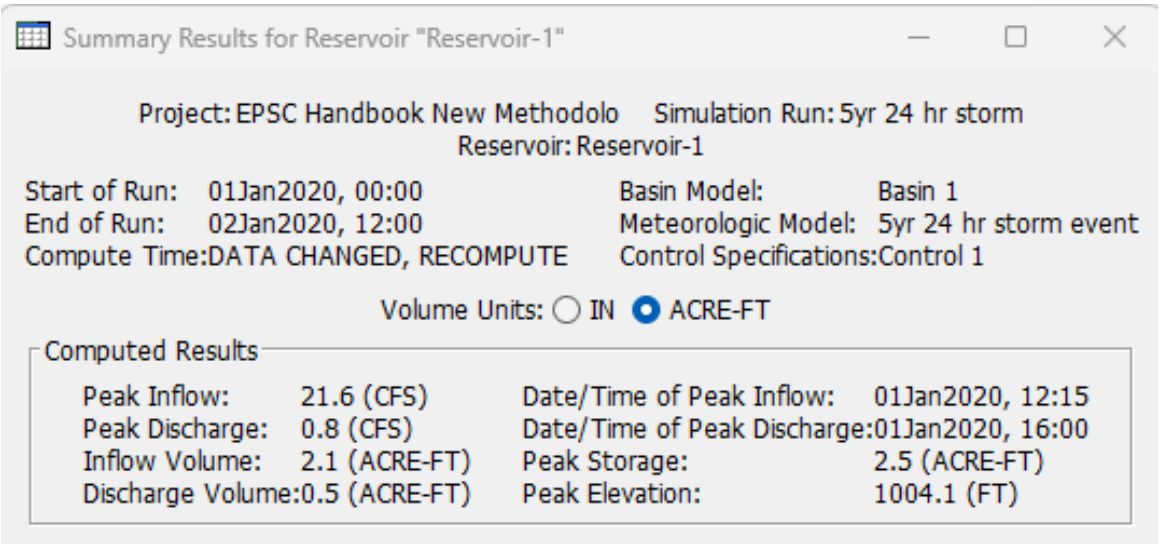


*Disclaimer – Common commercial software is not endorsed by the state, but may be and is commonly used by designers throughout the state.

An outlet pipe of 24 inches has adequate capacity.

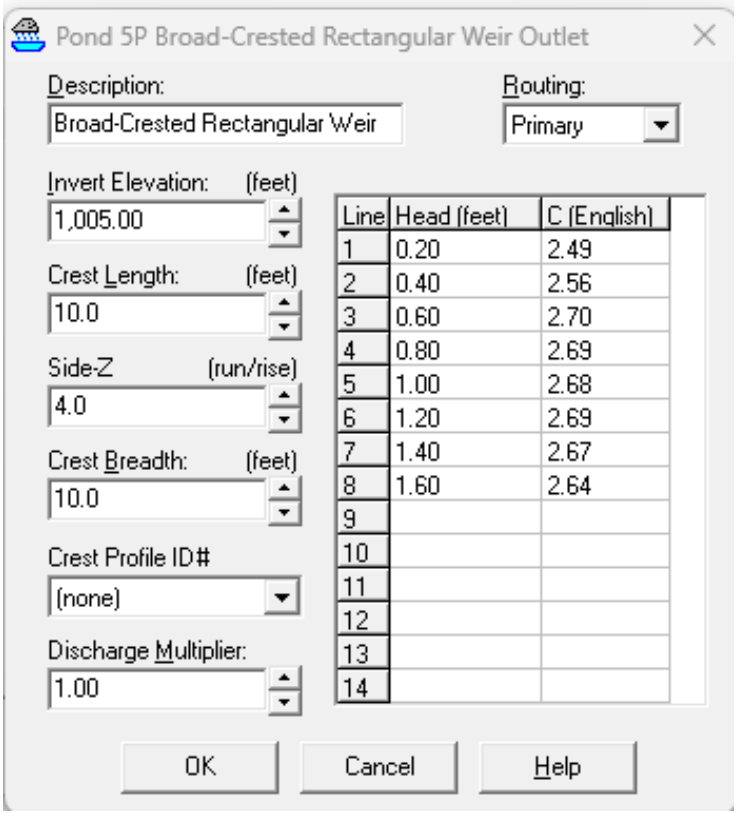
HEC-HMS is also shown as an example software for calculations below.





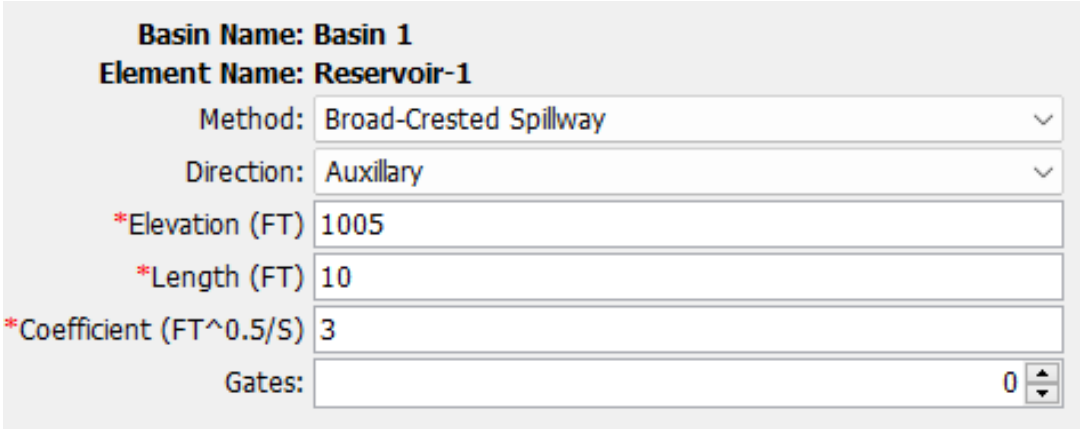
Step 10 – Design of the emergency spillway: Flow credit is given to the principal spillway but not the dewatering device. A minimum freeboard of 1 ft from the top of the principal spillway to the emergency spillway is necessary. Therefore, first try the emergency spillway at an elevation of 1005 ft. The emergency spillway will need to be checked with the 25-year, 24-hour design storm, as it is used to convey larger storm events. Use the following assumptions in HydroCAD as shown below:

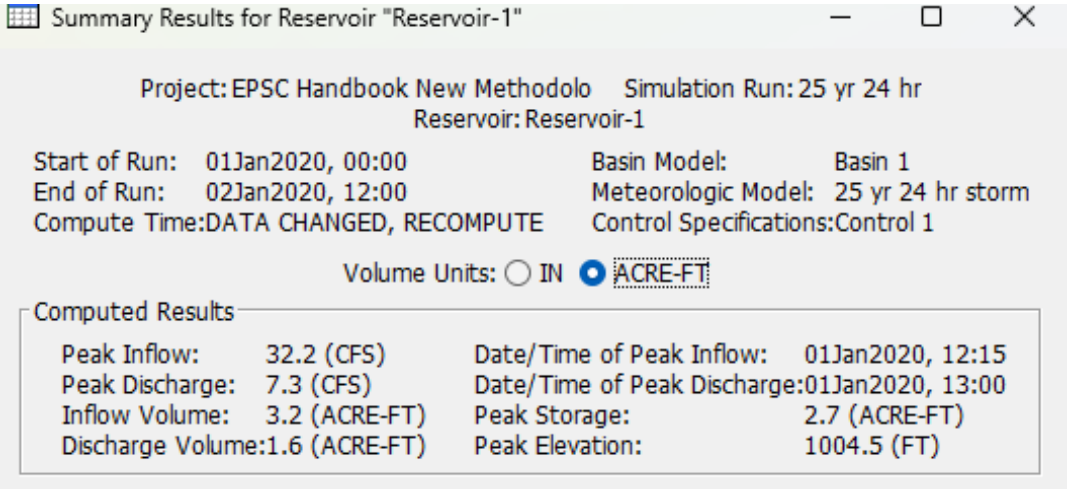
- Emergency spillway elevation = 1004.0’ (Principal Elevation) + 1’ (minimum freeboard) = 1005.0 ft;
- Emergency Spillway Side slopes = 4H:1V; and
- Assume a weir length = 10 ft.



*Disclaimer – Common commercial software is not endorsed by the state, but may be and is commonly used by designers throughout the state.

HEC-HMS is also shown as an example software for calculations below.





The emergency spillway elevation of 1005 ft is adequate. The 25-year, 24-hour storm event reaches an elevation of 1004.5 ft in the HydroCAD model. Therefore, the freeboard controls this elevation. The top of the berm elevation is 1006 ft, which is adequate since it exceeds the elevation of the emergency spillway elevation plus 1 ft of freeboard. The 50-year and 100-year, 24-hour storm events are checked with these elevations, and they are able to pass through the emergency spillway without overtopping.

Step 11 – Design of a skimmer: To understand the methodology when designing a skimmer or perforated riser, refer to Sections 4.4.12.2.1 or 4.4.12.2.2, respectively. However, for completeness, the example is continued here. The live storage volume of 47,264 ft^3 needs to be dewatered between 48 and 72 hours. Assume a dewatering time closer to 72 hours is ideal for the provided example. Determine the head on the skimmer orifice. The head will vary depending upon the manufacturer's design depth of submergence for a given orifice diameter. Use the following Skimmer size vs. orifice head table based on the Faircloth® skimmer as an example.

Skimmer Size (in)	Head (H) on Orifice (ft)
1.5	0.125
2	0.167
2.5	0.208
3	0.25
4	0.333
5	0.333
6	0.417
8	0.5

The flow through an orifice can be computed via Eqn 29 (Section 4.4.12.2.1), trying a 5-inch skimmer that has a driving head of 0.333 ft.

$$Q = N \times C_d \times A \times (2 \times g \times H)^{1/2}$$

$$33{,}824 \frac{ft^3}{72 \text{ hours}} = 1 \times 0.6 \times \frac{\pi \times d^2}{4} \times (2 \times 32.2 \frac{ft}{s^2} \times 5 \text{ in})^{1/2}$$

$$0.13 \frac{ft^3}{s} = 1 \times 0.6 \times \frac{\pi \times d^2}{4} \times (2 \times 32.2 \frac{ft}{s^2} \times 0.333 \text{ ft})^{1/2}$$

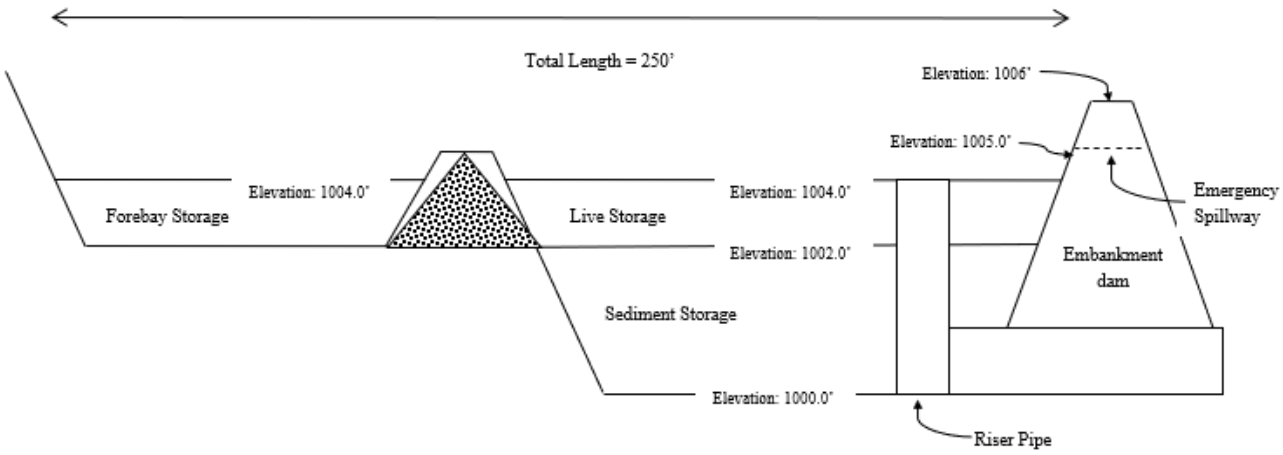
$$d = 0.244 \text{ ft} = 2.93 \text{ inch}$$

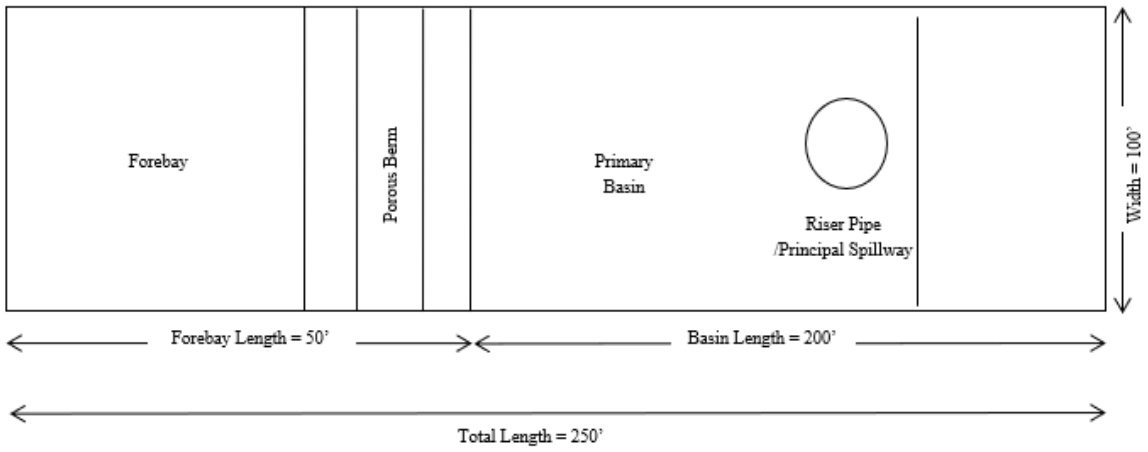
Therefore, a 5-inch skimmer with an orifice of 2.93 inches (or round up to 3 inches) is sufficient to dewater the 34,176 ft^3 in 72 hours.

Option: The above parameters can be entered into hydraulic routing software to refine the design.

Note: This sediment basin design example does not include emergency spillway lining calculations and design, nor does it include principal spillway outlet structure apron design and calculations. Examples regarding the lining of the emergency spillway can be found in Section 4.2.6.1, and outlet protection can be found in Sections 4.3.4, 4.36, and 4.3.7. Furthermore, the sediment basin design example does not discuss the anti-buoyancy pad of the principal spillway, anti-vortex and trash rack, anti-seep collars, details of surface skimmer connection to riser & skimmer base pad, or forebay berm design.

Below are schematics of the basin.





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4.4.8 Sediment Trap



Source: TDEC

Definition and Purpose

A sediment trap is a small, temporary basin created by excavating within a low area, creating an embankment, or both, to capture sediment-laden stormwater runoff. Its primary purpose is to temporarily detain the runoff, allowing suspended particles to settle before the runoff is released. Sediment traps are commonly paired with check dams, stone spillways, or other outlet controls that slowly release the water to reduce downstream erosion.

Appropriate Applications

Sediment traps, or their engineered equivalents (Chapter 5), are to be used where treatment of sediment-laden runoff is necessary, as specified by the Tennessee CGP. They are commonly installed at the outlets of stormwater diversion structures, slope drains, channels, or ditches where runoff is expected to carry high sediment loads (USEPA, 2021). Their placement is typically below disturbed areas, at low points such as swales, ditches, or near site perimeters where concentrated flows leave construction zones.

Limitations and Maintenance

A Sediment trap is not recommended for use in a drainage area greater than 10 acres. When a drainage area exceeds 10 acres, consider the use of a temporary sediment basin (Section 4.4.7) or a treatment train that may include multiple sediment traps (or a sediment trap in combination with other EPSC measures) in order to sufficiently treat the stormwater. Further, sediment traps have a relatively short lifespan, often functioning effectively for less than two years. Projects longer than two years should account for the complete

reconstruction of sediment traps. While these structures effectively capture coarse sediments, their detention times are not always sufficient to settle out finer particles such as silts and clays (USEPA, 2021). Sediment traps are to be removed once the contributing area has been permanently stabilized.

Proper maintenance is essential to ensure that sediment traps continue to function as designed. Sediment should be removed once it has accumulated to half the sediment storage volume. Removed sediment must be disposed of properly or stabilized onsite so as not to re-contribute to sediment loss. Regular inspections are critical to assess structural integrity and verify that the trap is draining properly. Further, check for erosion, seepage, or settlement of the embankment. Any displaced riprap, clogged filter stone, or damaged outlets should be repaired or replaced immediately. Embankments should be restored to design grade if settlement occurs, and all woody vegetation should be cleared from the structure to maintain stability. Once construction is complete and drainage areas are stabilized, the trap and all accumulated sediment needs to be removed, then the area backfilled and stabilized to prevent future erosion and sediment loss.

Planning and Design Considerations

Sediment traps are to be installed before land disturbance begins and located according to natural drainage patterns so that runoff from disturbed areas is directed into them. Placement should prioritize intercepting flow as close as possible to the source of sediment, while runoff from undisturbed areas would ideally be diverted around the construction. While applicable for larger drainage areas, to maintain trapping efficiency and structural stability, it is recommended to limit the drainage area contributing to a single trap to five acres (Line & White, 2001; USEPA, 2021). Traps are to be positioned where construction crews can readily perform inspections, sediment removal, and repairs without interfering with ongoing site activities (NCDEQ, 2013).

Design plans should address both performance and safety concerns. The designer and developer should be aware of the potential hazards that a temporary wet pond represents to the health and safety of a neighborhood, as specified in the sediment basin section (Section 4.4.7). Storage capacity is achieved through excavation, embankment construction, or both. To prevent embankment failure during extreme events, bypass outlets or stabilized emergency spillways should be incorporated, with flow directed toward stable, undisturbed areas. Traps are not to be located in wetlands, streams, buffer zones, or on steep, unstable slopes, as these conditions compromise effectiveness and increase risk of failure (VDEQ, 2024). Additional measures, such as outlet protection, porous baffles to improve flow distribution, and compost filter sock traps in suitable conditions, can enhance sediment retention performance.

The formal design and dimensions of a sediment trap should be documented in the EPSC plans. The minimum total storage volume of a sediment trap is to be 134 cubic yards (3,618 cubic feet) per acre of disturbed area draining to the trap. This volume, at minimum, is divided evenly between the sediment storage and live storage zones (see Section 4.4.7 for definitions and functions of these zones). It is recommended that a minimum length-to-width ratio of 2:1 be used to promote settling time and sedimentation. The spillway should be able to convey the design storm, and the live storage should be able to dewater between 48 and 72 hours. Embankments should not exceed five feet in height from the top of sediment storage to the crest of the berm; they are to be machine-compacted, with side slopes no steeper than 2H:1V, and have a width that corresponds to the associated drainage area for stability and discharge purposes (Table 4.4.8-A). Freeboard of at least one foot should be provided from the highwater elevation of the design storm and the crest of the berm. The emergency spillway will be used to pass the design storm (2-year, 24-hour or 5-year, 24-hour) corresponding to the entire drainage area, as the resiliency of the measure(s) is increasingly important as rainfall patterns change and more intense storm events occur. The sediment storage is excavated below the existing ditch bottom and serves as the primary sedimentation zone, while the live storage zone, measured from the top of the sediment storage zone to the invert of the overflow weir on the check dam, provides detention after rainfall (Figure 4.4.8-A). Sediment storage design dimensions can be figured from Figure 4.4.8-B.

Table 4.4.8-A: Recommended embankment widths for sediment traps with various drainage areas. Sources: Adapted from MPCA (2023) and Roberts (1995).

Contributing Drainage Area (ac)	Embankment Width (ft)
1	4
2	6
3	8
4	10
5	12
6	14
7	16
8	18
9	20
10	22

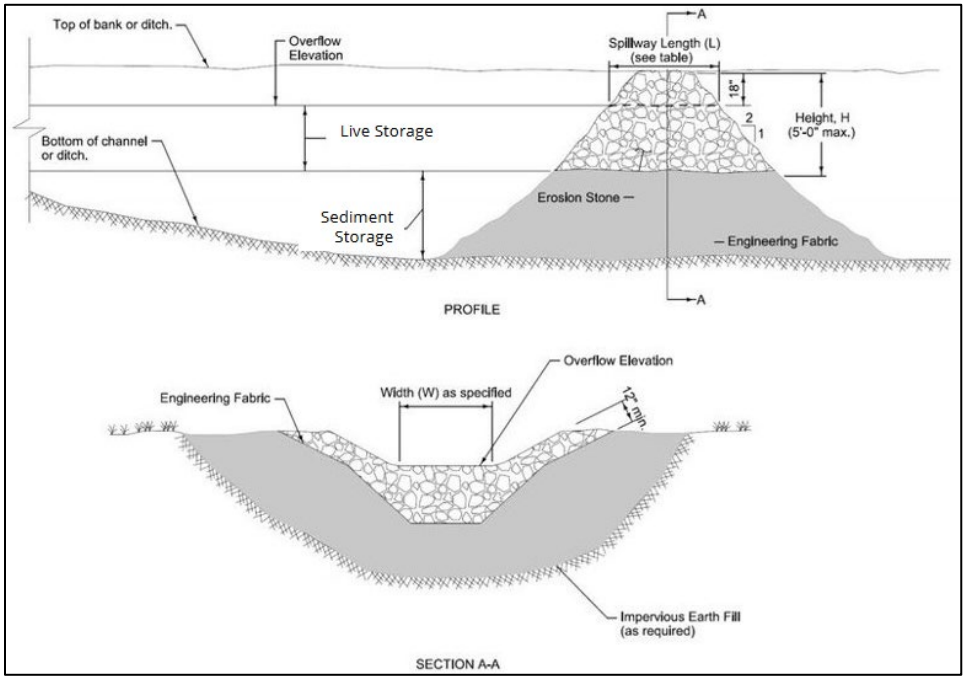


Figure 4.4.8-A: Sediment trap components. Source: Iowa Sudas (2019).

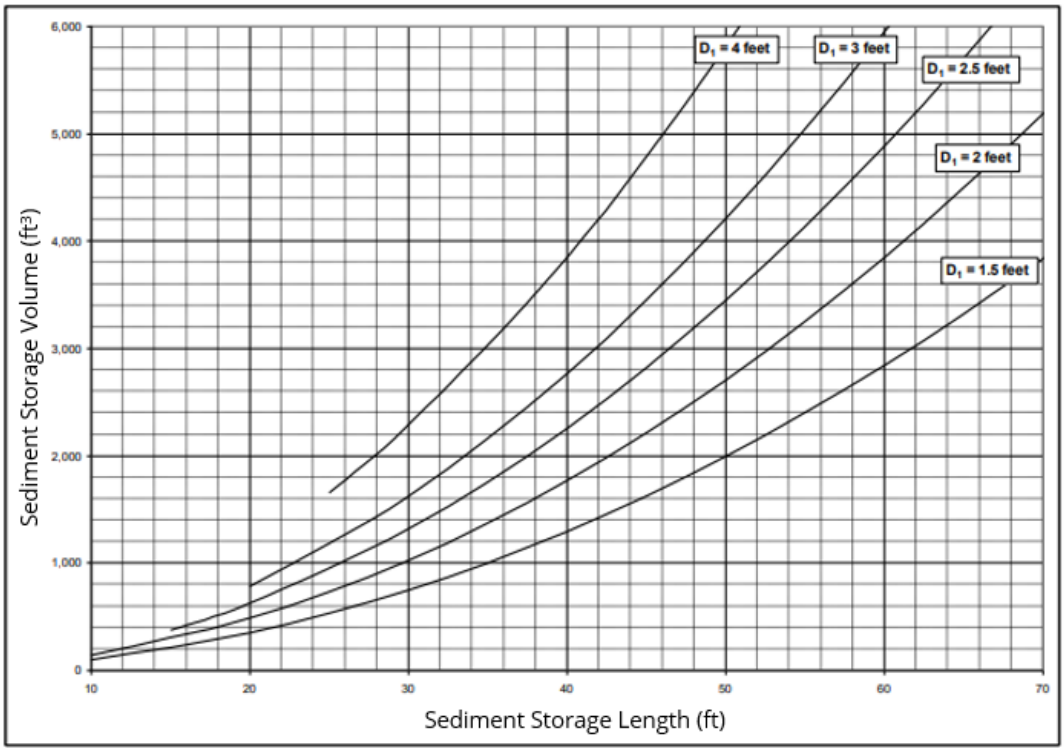


Figure 4.4.8-B: Sediment storage length (at the top of sediment storage) and depth dimensions based on sediment storage volumes. Source: TDOT.

The outlet of a sediment trap usually consists of a stone (conventional riprap) weir built into the embankment, which slows the rate and velocity of runoff and creates a temporary pool (Iowa SUDAS, 2019). This can be achieved through the construction of a riprap check dam, enhanced check dam, or gabion check dam (Figure 4.4.8-A). As such, follow the design recommendations in Sections 4.3.3 and 4.3.3.1. The height of the check dam is often between one and a half and three feet above the sediment storage elevation (TDOT), and the check dam should be able to convey the design storm. Though not necessary, TDOT recommends a two-foot offset between the storage basin and check dam, which is shown on the corresponding standard drawing. A geotextile fabric must be placed between the conventional rip rap and soil to prevent piping and erosion of the spillway(s). The width of the weir embankment (and earthen embankment) can be sized in accordance with the associated drainage area (Table 4.4.8-A). The crest of the outlet should be set at least one foot below the embankment to ensure controlled discharge.

To enhance settling efficiency, the trap should be designed with an elongated flow path, providing a minimum 2L:1W ratio, as previously mentioned. Porous baffles (Section 4.4.12.1.1) should be incorporated to reduce short-circuiting, particularly if high velocities are anticipated. Typically, at least two rows of porous baffles are recommended; refer to Section 4.4.12.1 for further details on porous baffles. In addition, a check dam, enhanced check dam, or gabion check dam may be used at the entrance of a sediment trap to create a treatment train and to decrease the maintenance of the trap itself. Refer to the forebay design of sediment basins (Section 4.4.7) for details regarding the use of check dams as a sediment basin/trap forebay. It is quicker and more cost-effective to remove sediment from a check dam than from the sediment storage zone of a sediment trap. Lastly, the condition that live storage surface area at capacity (acres) should be at least 100 times the peak inflow (cubic feet per second), as discussed for sediment basins (Section 4.4.7, Eqn 28), would be an ideal design consideration to improve settling within a sediment trap.

Sediment traps can include an excavated area that allows water to be detained and settling to occur, and a place for sediment to be stored. In this case, if the entire storage area was below grade, dewatering through pumping is likely necessary. When pumping, a skimmer (Section 4.4.12.2.2) and/or sediment filter bag (Section 4.4.12.6) should be used. The inflow or outflow should be treated with the proper mechanism. No erosion should occur from the pumps' discharge; appropriate energy dissipation (Section 4.3.4) measures should be used.

Sediment traps should remain in service until the drainage area is stabilized or replaced with a permanent measure. Once contributing areas are stabilized, the structure, including stone outlets and spillways, is to be removed, and the area is to be graded and stabilized in accordance with site plans.

Example Application

-Example courtesy of TDOT-

Given:

A drainage area of 2.5 acres that is fully disturbed, channel slope of 1.7% with a bottom width of 6 feet and side slopes of 4H:1V.

Determine:

The design for a sediment trap.

Solution:

Step 1 – Determine the required storage volume: The total storage volume is calculated as 3,618 cubic feet per acre of the 2.5-acre drainage area. Thus, the required storage volume is 9,045 cubic feet, where 4,522.5 cubic feet are designated to the live storage volume and sediment storage volume.

Step 2 - Determine the sediment storage design dimensions: To determine the length of the sediment storage area, L, a horizontal line corresponding to 4,522.5 cubic feet of storage is projected from the Y-axis of Figure 4.4.8-B. Inspection of the graph shows that the basin could be approximately 44 to 65 feet long, depending on the desired depth of sediment storage, D1. Based on the conditions of the site, it is decided that the basin can be constructed along a section of the roadway side ditch. This allows a trade-off between the depth of the basin and its length, and basin length at the top of sediment storage is determined to be 44 feet, so that the maximum sediment storage depth of 4 feet can be utilized. It is decided to use a 2:1 length-to-width ratio, and therefore, the sediment storage width is determined to be 22 feet. The sediment storage of the trap is assumed to be a rectangular prism with no side slopes. Thus, the sediment storage surface area at the top elevation (A1) is computed as 44 multiplied by 22 or 968 square feet.

Step 3 – Determine the height of a rock check dam: The storage volume offered by the live storage area is determined by D2, the height of the enhanced rock check dam spillway above the top of the wet storage. The live storage volume is computed as the average of A1 and A2 (live storage surface area at top elevation), multiplied by D2. A1 has been determined in the previous step. Thus, designing the dry storage area involves determining a spillway height such that the resulting value of A2 will produce a dry storage volume that is approximately half of the total required storage volume. Due to the number of variables, finding the spillway height is a trial-and-error process.

The goal is to use a minimum possible value of D2 to size the appropriate surface area at the top of live storage (A2). This will produce the most economic design in terms of land used to construct the sediment trap. However, this will yield the greatest maintenance burden. Knowing the side slope of the embankment, the live storage volume can be computed. If this volume is greater than required, no further computations will be required. Otherwise, D2 will be increased until an adequate live storage volume is obtained. During periods of high flow, water will collect in the live storage area until it spills over the weir built into the rock check dam. The minimum possible value of D2 can be computed as the product of the length across the top of the sediment storage (L) by the slope of the bed (S_0). This results in a minimum D2 value of 0.75 feet. This value is less than the minimum height of 1.5 feet; thus, an initial trial value of 1.5 feet for the minimum value of D2 will be used.

The volume of the live storage is a trapezoidal prism that extends from the upstream end of the sediment storage to the point at which the pool intersects the ditch bottom. Assuming the embankment has side slopes of 2H:1V, A2 would be 50 feet by 28 feet or 1400 square feet at a D2 of 1.5 feet. The volume of the trapezoidal prism can be approximated by averaging A1 and A2 and multiplying by D2, which yields 1701 cubic feet. This is less than the 4,522.5 cubic feet required. Thus, this process is repeated with larger D2 heights until an appropriate volume is identified, as shown in the table below:

D2 (ft)	A1 (ft^2)	A2 (ft^2)	Live Storage Volume (ft^3)
1.5	968	1300	1701.0
2	968	1456	2424.0
3	968	1792	4140.0
3.5	969	1972	5146.8
3.2	970	1863.04	4532.9

A D2 value of 3.2 feet yields a live storage of 4,533 cubic feet, which is just greater than the required storage. Thus, it is selected as the design value. It may be of importance to note that TDOT recommends a maximum D2 value of 3 feet while NCDEQ (2013) suggests a maximum D2 value of 3.5 feet. Other sources do not explicitly indicate a maximum weir impoundment height. Options to lessen the depth of the sediment storage and to create greater lengths and widths would be the next step if basing the design off of the greatest D1 did not yield an appropriate overall design.

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4.4.9 Sediment Tubes and Socks



Source: TNWRRC

Definition and Purpose

Sediment tubes (often referred to as wattles) and socks are sediment barriers often used in place of traditional barriers such as silt fence or conventional riprap in concentrated flow applications. They are used to slow the velocity of water, thereby allowing sediments to settle out, in addition to some filtering capacity.

Sediment tubes consist of weed- and seed-free straw or hardwood mulch wrapped in a tubular plastic, biodegradable netting, or similar encasing. Tubes should have sufficient density to hold their shape when saturated, but must also have open space to allow water to pass through. Sediment tubes are pre-assembled.

Sediment socks, often referred to as filter socks, filter logs, or compost socks, consist of flexible mesh tubes (preferably made from natural materials) filled with wood chips, composted material, or a combination of the two. Filter socks are constructed in the field with pneumatic equipment blowing the material into the casing, providing greater flexibility to obtain a sediment barrier of a specific length.

Appropriate Applications

Sediment tubes and socks are versatile perimeter and slope control practices that reduce runoff velocity, filter sediment, and capture pollutants in areas subject to sheet or rill erosion.

They are typically placed along slope contours, at the top or toe of slopes, around soil stockpiles or inlets, and along site perimeters, but can also function as check dams, diversion berms, or protection around sensitive trees and natural features where trenching is not feasible. They mostly function to retain sediment on site but can filter pollutants, particularly when composted fill is used. These measures are generally used in combination with other EPSC measures such as surface roughening, mulching, or seeding.

Limitations and Maintenance

These devices should never be used in streams and provide little to no benefit on pavement, rocky soils, or areas with heavy traffic where stakes cannot be properly anchored or where they may be damaged by vehicles. Correct installation along horizontal contours is critical; otherwise, flows can concentrate at low points, leading to bypass. Regular inspection helps to identify gaps, undercutting, or damage. Accumulated sediment is to be removed once it reaches one-half the device height, and repairs or replacements should be made promptly to maintain functionality. Over time, wattles and socks degrade, with straw wattles generally lasting 12 to 18 months and socks lasting six months to a year, necessitating a replacement schedule for longer projects (VDEQ, 2024). Nonbiodegradable tubes and socks must be removed and disposed of properly after site stabilization. Any trenches or depressions that were created during installation or removal need to be backfilled. Fully biodegradable options may be left on-site once construction is complete. Consider removal operations, as saturated tubes and socks can become heavy and difficult to move. Whenever feasible, use plastic-free alternatives as they promote more environmentally- and wildlife-friendly conditions and are biodegradable (can be left in place once construction operations are complete).

Planning and Design Considerations

Both sediment tubes and filter socks require careful preparation of the installation area to ensure effectiveness. The ground should be cleared of vegetation, debris, stones, and irregularities to maintain continuous contact between the surface and the device to prevent undercutting. For sediment tubes, shallow trenching, typically at least two inches, is needed to securely anchor them in place in addition to stakes. Wooden stakes are ideal for tubes, driving them in at an angle on the downstream side of the tube or driving two in on either side to form an A-frame. Install wooden stakes at four-foot intervals (shorter if applied in concentrated flows), leaving only a couple of inches of stake exposed above the wattle. This method of staking locks the wattle in place, preventing lateral movement and floatation. Avoid staking through the wattle as this will loosen the filling material, forming a preferential flow path, which may potentially lead to floating and undercutting. On the other hand, sediment socks generally do not require trenching, but staking is advised. It is especially important not to stake through sediment socks.

Sediment tubes may contain straw, which must be certified weed-free, or other approved media, or mulch. Sediment socks are typically filled with compost or coarse organic blends, with specifications provided in Table 4.4.9-A, compacted pneumatically for consistency and performance. Refer to Section 4.2.6.2 for guidance on recommended compost products. Filter socks can be assembled on-site by filling mesh sleeves to the desired length or delivered prefilled for immediate placement. Because socks can be assembled to various lengths (generally up to 250 feet), they offer flexibility in coverage across long slopes or perimeters. Sediment tubes, by contrast, are generally shorter and require segments to overlap at least 24 inches to eliminate the likelihood of bypass. Longer-duration projects may require periodic replacement.

Table 4.4.9-A: Minimum filler specifications for compost sediment sock filling. Source: TDOT.

Property	Standard
pH (pH Units)	5 - 8.5
Moisture Content (% Wet Weight)	< 60
Organic Matter Content (% Dry Weight)	25 - 100
Physical Contaminants (% Dry Weight)	< 1
Particle Size (% Passing)	2 inch - 99%; 3/8 inch - 30% to 50%

The selection of diameter and length for sediment tubes and filter socks depends on site slope, drainage area, and flow conditions. On slopes, both devices should be installed along contours with their ends turned upslope to prevent bypassing. Filter socks are generally available in diameters of eight, 12, 18, and 24 inches, with larger diameters suited for steeper or longer slopes. Similarly, sediment tubes range from 8 to 24 inches on slopes, but should have a minimum of 20 inches when installed in ditches to function effectively as velocity control devices.


Source: TDEC

Spacing between rows is based on slope gradient and length (Table 4.4.9-B). The drainage areas to only one row of sediment tube or similar should not exceed the maximum allowable load; a best practice is to have roughly one-quarter acre of drainage area per 100 linear feet for both measures (TDOT). In ditch or channel applications, both practices must extend up the side slopes high enough to prevent overtopping. Because individual segments are rarely long enough to span a ditch, overlapping or staggering joints are required to avoid gaps. For

filter socks, stacking smaller-diameter units can substitute for a single larger sock to meet height requirements. Refer to Figure 4.3.3-B for spacing requirements for sediment tubes and socks in concentrated flow applications.

Table 4.4.9-B: Horizontal spacing recommendations for sediment tubes and socks on slope applications. Source: TDOT.

Slope	Tube Diameter (inch)			
	8	12	18	24
2%	70	80	-	-
5%	30	60	80	-
10%	20	30	70	80
6H:1V	-	20	40	55
4H:1V	-	20	30	30
3H:1V	-	-	20	25
2H:1V	-	-	20	20

- Spacing not recommended to exceed 80 feet.

Finally, the design must consider flow volumes and project-specific water quality concerns. Sediment tubes and filter socks are not sized to convey large storm events; rather, they act as velocity reducers and sediment barriers, with filtration occurring primarily during smaller flows. This makes contour placement and spacing critical to avoid concentrating runoff between devices. In addition, designers should weigh nutrient considerations when using compost-filled socks, especially near sensitive or nutrient-impaired waters, as compost can both retain and release nutrients depending on site conditions. Thoughtful design that accounts for slope, drainage, material selection, and water quality ensures that both sediment tubes and filter socks function effectively within an erosion and sediment control system.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

TDOT. *Drainage Manual Ch10*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.4.10 Silt Fence and Associated Variants



* SF * SF * SF * SILT FENCE
* ESF * ESF * ESF * ENHANCED SILT FENCE
* SFB * SFB * SFB * SILT FENCE WITH WIRE BACKING

Source: TNWRRC

Definition and Purpose

Silt fences are temporary sediment control devices commonly used on construction sites to retain soil on disturbed land and prevent sediment-laden runoff from entering storm drains or nearby water bodies. They consist of a porous geotextile fabric supported by wood or metal posts and are typically installed to intercept sheet flow, reduce runoff velocity, and promote sediment deposition through settling. Variants such as silt fence with wire backing include added support using a wire-reinforced geotextile, making them more durable and suitable for areas adjacent to streams, wetlands, or steep slopes. Super silt fences are reinforced systems (typically chain link fencing and galvanized poles) designed for use where standard or wire-backed silt fences cannot meet slope length requirements or where space limits the installation of sediment basins or traps (VDEQ, 2024). Enhanced silt fences, featuring high-strength woven monofilament geotextile fabric and metal reinforcement, may be more ideal when the site has various sensitive areas such as sinkholes, ETWs, etc. These systems offer superior tensile, puncture, and hydraulic performance compared to standard silt fences, allowing for greater filtration capacity and durability under high-flow conditions (TDOT).

Appropriate Applications

Silt fence, silt fence with wire backing, and super silt fence are only to be used in areas of sheet flow and installed on contour (NCDEQ, 2013). Common examples include at the toe of a slope, along the perimeter of a construction site, along the banks of ditches or swales, and around the perimeter of a stockpile.

Enhanced silt fence can be used in areas of sheet flow and is the only variant of silt fence applicable in areas of concentrated flow. It can be used as a check dam alternative in small open channels, swales, and ditches; however, they are not to be used in streams (TDOT). In sheet flow applications, this variant is particularly useful in areas draining unstable watersheds, where sediment loads are expected to be elevated, where runoff enters ETWs, or during vegetation establishment. For check dam alternatives, refer to the specifications in the standard drawing. Herein, the enhanced silt fence will be discussed in its sheet flow applications only.

Limitations and Maintenance

Silt fence, silt fence with wire backing, and super silt fence do not provide enough durability for use in areas of concentrated flow. The forces associated with concentrated flows will lead to blowouts, undermining, and bypass. Further, maintenance of silt fence is imperative to prevent blowouts, overtopping, and structural failure. Once sediment has accumulated to a height of half the silt fence, it is to be removed and disposed of properly. Disposal may include keeping the sediment onsite and stabilizing it. Regular inspection is essential: fences should be inspected regularly to identify sagging, damage from construction equipment, or signs of undermining along the toe.


Source: KTC (2015)

Maintenance challenges are often compounded by site conditions. Saturated soils behind the fence can make it difficult to remove sediment with machinery, particularly on linear construction sites with limited space for reinstallation. In such cases, an alternative approach is to leave the sediment in place if it is stable and install an additional silt fence to continue capturing runoff (USEPA, 2021). However, if a fence consistently fails, this typically indicates improper installation or poor placement. In these situations, installing different EPSC measures may be more beneficial than reinstalling the compromised fence. Ultimately, silt fences must remain in place until disturbed areas are permanently stabilized, and final removal should include restoration of the site through grading and seeding of remaining sediment deposits.


Source: USEPA (2021)

Fabric degradation is another limitation; the geotextile material may wear or clog with fine sediment over time, which significantly reduces its permeability and effectiveness. In such cases, the fabric must be replaced, or in some instances, an entirely new silt fence should be installed either upslope or downslope of the original location. While support posts and reinforcing wires can often be recycled or reused, traditional silt fences are one-time-use plastics that contribute microplastics to the environment as they degrade (Whitman et al., 2025). In addition to environmental concerns, silt fences block the passage of small animals and amphibians, posing wildlife concerns.

Planning and Design Considerations

Silt fence and its variants are systems to retain sediment on the construction site by temporarily ponding sheet flow and promoting deposition. Its primary form of sediment treatment is by settling sediments through ponding, though some geotextile materials may provide secondary benefits of filtering sediments. They are applicable under differing scenarios (Table 4.4.10-A), and therefore fabric and reinforcement requirements of the variants also vary (Table 4.4.10-B). Use 14-gauge wire backing and a maximum mesh spacing of six inches for wire-backed silt fence. For super silt fence, use chain link is galvanized No. 11.5 Ga. steel wire with 2.25-inch opening, No. 11 Ga. aluminum coated steel wire in accordance with ASTM A 491, or galvanized No. 9 Ga. steel wire top and bottom with galvanized No. 11 Ga. steel intermediate wires (VDEQ, 2024). One fabric requirement differentiation is the flow through the material due to ponding. The designer should anticipate ponding and provide sufficient storage areas and overflow outlets to prevent flows from overtopping the fence. Silt fence should be installed on contour and curled uphill on each end of the fence in a “J-hook” pattern to prevent end flow and scouring out. This J-hook pattern may also be adopted when site conditions prevent the placement of silt fence on contour. Another option, when using multiple rows of silt fence, is to connect the rows with a longitudinal strip of silt fence to prevent sediment-laden runoff from migrating around the rows of silt fence. Provide stabilized outlets to protect the fence system and release storm flows that exceed the design storm.


Source: TDEC


Source: TNWRRC

Table 4.4.10-A: Maximum slope length (feet) upgradient from silt fence. Sources: PADEP (2012) and VDEQ (2024).

Slope	Silt Fence	Silt Fence with Wire Backing	Super Silt Fence
≤ 2	100	500	1000
5	100	250	550
10	50	150	325
15	35	100	215
20	25	70	175
25	20	55	135
30	15	45	100
35	15	40	85
40	15	35	75
45	10	30	60
50	10	25	50

Note: Guidance on enhanced silt fence is not provided. It can be assumed to have similar allowable slope lengths as a super silt fence.

Table 4.4.10-B: Physical property requirements of silt fence fabrics. Source: TDOT.

Physical Property	Test	Silt Fence	Silt Fence with Wire Backing / Super Silt Fence	Enhanced Silt Fence
Grab Str. Tensile (lbs) - Wrap Direction	ASTM D 4632	≥ 120	≥ 310	≥ 370
Grab Str. Tensile (lbs) - Fill Direction	ASTM D 4632	≥ 100	≥ 200	≥ 230
Elongation at Failure (%)	ASTM D 4632	≤ 20	-	-
Burst Str. (PSI)	ASTM D 3786	≥ 250	≥ 400	≥ 470
Puncture Str. (lbs)	ASTM D 4833	≥ 60	≥ 105	≥ 110
Trap. Tear Str. (lbs) - Wrap Direction	ASTM D 4533	≥ 50	≥ 100	≥ 115
Trap. Tear Str. (lbs) - Fill Direction	ASTM D 4533	≥ 40	≥ 60	≥ 75
Apparent Opening Size (U.S. Sieve)	ASTM D 4751	#30 to #70	#70 to #100	#30 to #80
Flow Through Rate (gal/min/sq.ft)	ASTM D 4491	≥ 4	≥ 18	≥ 110
Min Ultraviolet Residual (%)	ASTM D 4355	≥ 70	≥ 90	≥ 90

Silt fence, regardless of type, can be installed in one of two ways: static slicing or trenched, both of which require staking the fence into the ground and affixing the fence to such stakes (Table 4.4.10-C). The static slicing method, often preferred for its superior soil sealing and strength, uses specialized slicing machines equipped with a narrow vertical blade. As the

blade is pulled through the soil, it creates a slit (approximately 12 inches deep) and simultaneously inserts the silt fence fabric directly into the ground behind it [in which the depth of buried fabric depends on the type of silt fence (Table 4.4.10-C)]. The slicing blade slightly lifts the soil to minimize horizontal disturbance while allowing for efficient vertical compaction. After insertion, a tractor or machine-mounted roller can be used to compact both sides of the fence line by driving over the disturbed soil multiple times, creating a tight seal. This vertical compaction minimizes air gaps and potential seepage paths, resulting in a more stable and longer-lasting installation. Once the fabric is in place, posts and the fabric are securely attached using wire ties, staples, or lath, depending on post type (Table 4.4.10-C; TDOT; USEPA, 2021). In the trenching method, installation begins by excavating a shallow trench, typically four to six inches deep and wide. The bottom edge of the filter fabric is placed into this trench, allowing at least six inches of fabric to be buried, depending on silt fence type (Table 4.4.10-C). After placing the fabric, the trench is backfilled with excavated soil and compacted firmly, either manually or with mechanical equipment, to create a seal that prevents water from flowing underneath the barrier. Posts are then driven into the ground on the downstream side of the fabric, where post material, length, and depth driven into the ground depend on the variant of silt fence (Table 4.4.10-C). If the post material is metal, a post cap is required. The fabric is then attached to the posts using wire ties, staples, or wood lath, ensuring it is tight and upright (TDOT; USEPA, 2021; VDEQ, 2024). While trenching is a widely used and cost-effective method, proper compaction of backfill soil is critical: inadequate sealing can lead to undercutting and system failure, especially in saturated or high-flow conditions. Compared to static slicing, trenching may offer more flexibility in maneuvering around obstacles, but requires greater attention to soil sealing and reinforcement and time (USEPA, 2021).



Photo courtesy of J. R. Buchanan

Table 4.4.10-C: General installation guidelines for silt fence and silt fence variants. Sources: [a]TDOT and [b]VDEQ (2024).

Feature	Silt Fence[a]	Silt Fence with Wire Backing[a]	Super Silt Fence[b]	Enhanced Silt Fence[a]
Primary Use	Sheet flow, sediment-laden runoff	Higher stress, steep-sloped areas	Higher stress, space-constrained areas	Higher stress, steep slopes, Concentrated flow*
Support Structure	Fabric only, on posts	Wire mesh backing plus fabric on posts	Chain link fence plus high-strength fabric	Wire mesh backing plus high-strength fabric
Post Material	Steel or wood	Steel	Galvanized or aluminum	Metal-studded tee
Post Spacing	Max 6 feet	Max 4 feet	Max 10 feet	Max 2 feet
Post Depth into Ground	Min 20 inches	Min 36 inches	Min 24 inches	Min 42 inches
Post Length	Min 48 inches	Min 64 inches	Min 57 inches	Min 84 inches
Post Dimensions (nominal)/Weight	2.25 x 2.25 inches	1.25 lbs/ft	2.5-inch diameter	1.25 lbs/ft
Fabric Height Above Ground	Min 26 inches	Min 26 inches	Min 32 inches	Min 40 inches
Fabric Height Below Ground	Min 6 inches	Min 6 inches	Min 8 inches	Min 8 inches
Silt Fabric Width	36 inches	Min 32 inches	Min 40 inches	Min 48 inches
Fabric Attachments	Tied/stapled to posts	Tied	Tied or clipped to a chain link fence	Tied
Installation Method(s)	Static slicing or trenching	Static slicing or trenching	Trenching	Trenching
Trench Depth	4-6 inches	4-6 inches	8 inches	Max 14 inches
Trench Width	4-6 inches	4-6 inches	6 inches	Max 18 inches
Trench Backfill	Required, compacted	Required, compacted	Required, compacted	Required, compacted
Removability	Easy	Easy to moderate	Moderate to difficult	Moderate to difficult

*When using enhanced silt fence in areas of concentrated flow, refer to the proper standard drawings. Herein, specifications of the enhanced silt fence is for sheet flow.

For maximum effectiveness, silt fences should be installed along the contour of the land. Placement near ridges or drainage divides, where water movement is minimal, provides little benefit and may even introduce unnecessary erosion or sedimentation problems. Further, installation on upgradient property boundaries likely has a negative impact as it will only impound water running onto the site. Installation must ensure the bottom of the fence sits on a nearly level grade (0%, ± 0.5%) and follows the natural ground contour as closely as possible. At the base of slopes, fences should be installed five to seven feet away from the toe to provide space for water ponding and sediment accumulation. Silt fences can be installed with or without backing. For fences without backing, the drainage area should not exceed one-quarter acre per 100 feet of fence, where the drainage area initiates at the lowest elevation of silt fence (i.e., not at the end of a roll of silt fence). When using backing, the allowable drainage area increases to one acre per 150 feet of fence. Given the typical lifespan of plastic fencing is six to 12 months, projects lasting longer than a year should plan for full replacement, budgeting for material replacement approximately every nine months.

Summary points and general good installation techniques include:

- Ensure that the height of the sediment fence extends roughly 26 inches above the ground surface. Fences exceeding this height may impound sufficient volumes of water resulting in failure of the structure;
- Construct the filter fabric from a continuous roll cut to the length of the barrier to avoid joints. When joints are necessary, securely fasten the filter cloth only at a support post with a minimum of four feet overlap to the next post, or roll the fabric together and fasten to one post to create a stronger joint. Where joints are necessary, plan the roll layout so as not to have joints at low points;
- Do not attach filter fabric to trees;
- When silt fence is installed adjacent to streams, wetlands, or other natural resources, a wired-backed silt fence is recommended;
- Install posts at the specified depths and spacings on the downstream side of the silt fence, and as close as possible to the fabric, enabling posts to support the fabric from upstream water pressure;
- Securely attach the silt fence fabric to the posts on the upstream side of the posts. For steel posts, attach fabric to the posts using wire or plastic zip ties with a minimum 50-pound tensile strength, at least five to a post. Three ties should be installed in the upper eight inches for top strength. Ties should be installed on the diagonal, as opposed to on the horizontal, to anchor more strands of fabric to the post. For hardwood posts, attach fabric with 17-gauge wire staples (3/4" wide x 1/2" long), at least five to a post. Three staples should be installed in the upper eight inches for top strength; and
- Install J-hooks where the silt fence cannot be completely installed on the contour.

Traditional, synthetic silt fence typically removes 80 to 90 percent of sediments (Faucette et al., 2008; Perez et al., 2024). However, agencies across the nation indicate plastic silt fence is one of the most, if not the most, problematic practices for contributing plastics (micro and macro) to the environment, and the need for a plastic-free alternative to silt fence. Though the natural silt fence counterpart may not yet be readily available, it is in testing (see associated picture). Wattles absent of plastic nettings and proper installation techniques (Section 4.4.9) as well as slash mulch berms (Section 4.4.2) are viable, environmentally- and wildlife-friendly alternatives. These alternatives have demonstrated similar sediment removal performance (up to 98%) as synthetic silt fence (Donald et al., 2013; Schussler et al., 2021; Perez et al., 2024).



Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

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4.4.11 Weep Berms



Source: Warner et al. (2012)

Definition and Purpose

A weep berm is an earthen structure constructed perpendicular to the direction of runoff, designed to control stormwater runoff and improve water quality. The berm temporarily stores runoff, allowing sediments and sediment-bound pollutants to settle before discharging runoff to the downgradient side of the berm, usually to a grassed or riparian area. This slow "weeping" release enhances the treatment capacity of the buffer by extending the residence time and contact time with vegetation and soils.

Appropriate Applications

Weep berms are best suited for locations downgradient of disturbed areas and located on contour such that runoff can be temporarily stored and treated without causing damage to the berm or surrounding land. Further, they may be located off contour when diversions redirect runoff to the weep berm (Warner et al., 2012). They are commonly placed along linear developments such as haul roads, pipelines, and transmission lines, as well as below stockpiles and cut or fill slopes. These structures are most effective in flatter drainage areas with slopes under 10%, where they can slow runoff and promote infiltration and settling; on larger sites, multiple berms may be installed to divide the drainage area and prevent excessive flow to any single structure (MPCA, 2023).

Limitations and Maintenance

Weep berms are not to be placed in streams or wetlands. Further, it is recommended that they are not placed in ditches or swales unless the earthen berm is stabilized with a rolled erosion control product. Accumulated sediment must be removed and disposed of properly or stabilized once half the storage capacity is reached. Periodic mowing may also be necessary to manage vegetation. Lastly, inspect outlet pipes for clogging and clear debris when necessary.

Planning and Design Considerations

Weep berms are a space-efficient EPSC measure and may be a viable alternative when sediment basins or traps are infeasible. For disturbed areas greater than one acre, two or more berms or intermediate dikes should be used in series, spaced so that the toe of the upstream berm aligns with the crest of the next downstream berm (GSWCC, 2016). When sized properly, weep berms are capable of completely capturing, infiltrating, or detaining, and treating runoff from small storms and a sizable portion from larger storms. Warner et al. (2012) advocated that weep berms typically remove 90% of sediments in addition to the removal capabilities of nutrients and bacteria. However, the authors also noted that performance can be influenced by the types of soils used to construct the berm (infiltration capacity), sediment yield of the drainage area, and the type of weep berm used.

Generally, two types of weep berms are considered: contour and gradient. Contour weep berms are built along contours of equal elevation, with their ends curving upslope. This design gives them a horseshoe-shaped storage area and, when properly installed, allows them to blend naturally into the surrounding landscape. On longer stretches, earthen dikes may be placed at intervals to divide the berm into storage cells, reducing risk by ensuring that failure of one cell does not release the full volume of stored runoff. Gradient weep berms, by contrast, are typically used in combination with diversions or sediment ditches. They often include check dams along their length, which provide temporary runoff detention. During heavier storms, these check dams slow the flow of water, promoting infiltration and allowing more sediment to settle out before runoff continues downstream (Warner et al. 2012).

When designing a contour weep berm, the berm is to be sized to provide adequate runoff storage while accounting for the extent of the disturbed area. Berm length is typically based on the area of disturbance, with longer berms built at lower heights and shorter berms constructed taller. For large drainage areas, multiple berms may be placed in series to create a larger storage capacity. The berm height is generally set to contain runoff from the 5-year, 24-hour storm, while also accommodating additional storage for sediment when needed. This creates a tradeoff between the overall berm size and the frequency of required

sediment clean-out. Outlets are set at elevations that correspond to the top of the sediment storage zone. Access should be provided for the maintenance and removal of deposited sediments. Design guidelines also suggest that weep berms should not exceed four feet in height and should have a minimum top width of 12 inches.

A gradient weep berm consists of a trapezoidal channel with check dams placed at intervals along its length and outlets located on the downgradient slope. The channel dimensions, including bottom width, side slopes, and overall slope, are selected based on site conditions. Check dams regulate runoff detention, with their crests designed to store runoff from the design storm while also accounting for sediment storage. Spacing of check dams is important for volume control; they are typically positioned so that the crest of each downstream dam is at the same elevation as to toe of the upstream dam. As with contour berms, outlet pipes should dewater between 48 and 72 hours.

For both types, total storage volume guidance is 3,618 cubic feet per acre of disturbed area draining to the berm, which is divided evenly between the sediment storage and live storage zones. The berms must be stabilized, either through compaction, vegetation, or a rolled erosion control product, before land disturbing activities commence. Outlets for weep berms are typically limited to straight pipes or rock lenses (Figure 4.4.11-A), with the invert set to store 1,806 cubic feet per acre of disturbed area for sediment storage. Clean-out markers should be placed at intermediate dikes to track when sediment removal is required. The outlet type, size, and spacing control the discharge rate. One option is to install outlets during berm construction, though this requires care to avoid crushing pipes when compacting. Alternatively, outlets can be installed by excavating trenches after the berm has been completed and then backfilling and compacting the soil around them, or by driving a steel pipe with a conical end through the berm to create openings for PVC outlets. To prevent embankment failure during extreme events, bypass outlets or stabilized emergency spillways should be incorporated, with flow directed toward stable, undisturbed areas. The spillway will be used to pass the design storm (2-year, 24-hour or 5-year, 24-hour) corresponding to the entire drainage area, as resiliency of the measure(s) is increasingly important as rainfall patterns change and more intense storm events occur. A riparian buffer, either grassed or forested, is a critical downstream component that aids in infiltration and filtering of released water.

Weep berms are generally built using common on-site construction equipment such as skid steers, backhoes, or track hoes. Construction begins with clearing vegetation from the berm footprint while protecting surrounding areas, since infiltration efficiency depends on minimizing soil compaction both up- and down-gradient of the structure. The berm itself should be compacted enough to ensure stability but still allow seepage, with soils containing

at least 10% clay and 20% combined silt and clay considered most suitable. Berms are typically installed in six to nine-inch lifts. For structural strength, compaction can be accomplished by tracking with a dozer, running a skid loader with a full bucket, or applying pressure with the bucket of a backhoe. A minimum of 90% standard Proctor density is often recommended to achieve the required balance between strength and permeability (Warner et al., 2012).

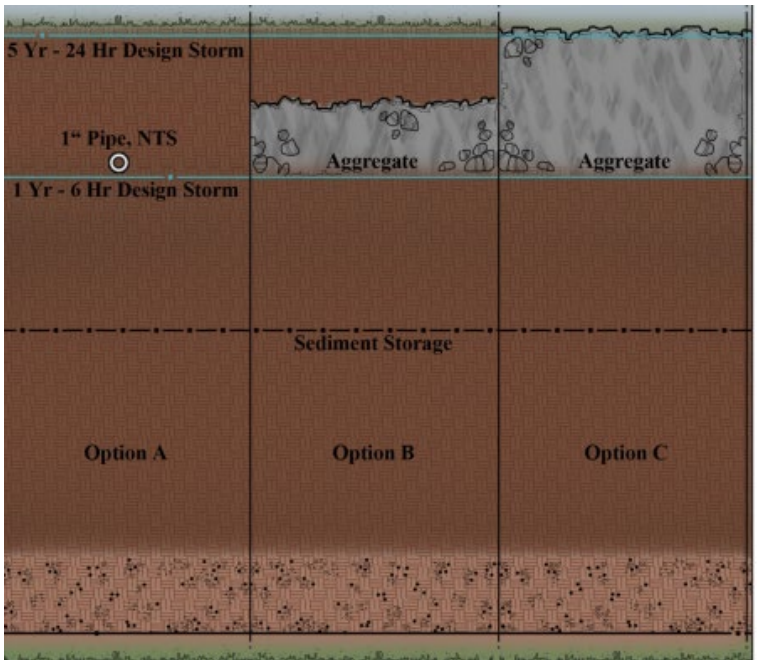


Figure 4.4.11-A: Weep berm outlet options. Source: Warner et al. (2012).

Example Application

-Example amended from Warner et al. (2012)

Given:

The land slope upgradient of the planned location of the weep berm is 2 percent with a drainage area of 4 acres that is fully disturbed. Soils at the project site are Bluegrass-Maury silt loam, which places them in HSG B. For sediment storage requirements, she needs 67 cubic yards per acre of disturbed land. The 5-year, 24-hour, which is the design storm, rainfall depth is 3.8 inches.

Determine:

a.) The berm height to storage volume relationship for a 2 percent slope and a 500-foot berm length;
b.) The sediment storage and live storage requirements for the weep berm; and
c.) The runoff volume associated with the five-year 24-hour design storm.

Solution:

a.) Assume the deposited sediment will form a triangular wedge, the watershed slope is constant, and the interior weep berm slope is 1.5:1 (height to volume). The table below shows the volume of sediment that could be stored behind a 500 ft long weep berm of varying berm heights based on the storage geometry. This can be used to determine when sediment removal is necessary.

Weep Berm Height (ft)	Storage Volume (ac-ft)
0.5	0.0739
1	0.2956
1.5	0.665
2	1.1823
2.5	1.8473
3	2.6601
3.5	3.6207
4	4.7291

b.) The given sediment storage requirement is 67 cubic yards per acre of disturbed land. For 4 acres, 268 cubic yards, or 0.166 acre-feet is required. Use the above table to determine the associated weep berm height for a sediment storage volume of 0.166 acre-feet. From linear interpolation, a height of 0.71 feet is determined. This height sets the outlet elevation. The live storage requirement is also 0.166 acre-feet. Therefore, the top of the live storage is 1.42 feet. Ensure the 0.166 acre-feet of live storage can be drained in 48-72 hours utilizing the orifice equation from Section 4.4.12.2.1 (Eqn 29).

c.) The CN for a newly graded Bluegrass Maury silt loam (HSG B) is 86. For a 3.8-inch rainfall depth over 4 acres, the associated runoff volume is 0.789 acre-feet (Eqns 1, 2, and 3). Linearly interpolating from the table, his corresponds to a height of 1.62 ft, which sets the height of the emergency spillway. Utilize a peak flow rate associated with the design storm (not enough information provided in the example to compute this) and the weir equation (Eqn 17 or 18, Section 4.3.3) or software such as HEC-HMS or HydroCAD, as shown in Step 10 of the example in Section 4.4.7, to appropriately size the emergency spillway.

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4.4.12.1.1 Porous Baffles



Source: City of Franklin

Definition and Purpose

Porous baffles are flow-distributing structures made from highly permeable materials, installed perpendicular to flow within sediment basins or traps. Their primary function is to reduce water velocity and turbulence, to minimize short-circuiting (phenomena in which influent water bypasses the intended flow path) by promoting uniform flow across the full width of the basin. This improved hydraulic performance enhances the settling of sediments by increasing residence time (Schussler et al., 2022).

Appropriate Applications

Baffles can be installed in any sediment trap or sediment basin. Porous baffles are often used when excessive short-circuiting or dead zones are observed. Furthermore, porous baffles establish an ideal location to place flocculants and polymers (Section 4.4.12.5) when chemical treatment is necessary.

Limitations and Maintenance

Ensure porous baffles are properly secured along the cross-section of the basin and extend the full width of the basin. The material should be taught. Porous baffles may increase the frequency with which accumulated sediment needs to be removed from the basin, particularly within the first chamber. Inspect for scour along the sides of the baffle and replace baffle material if it is damaged or degraded.

Planning and Design Considerations

Various types of materials can be used for porous baffles, including coir erosion blankets, wood excelsior, other RECPs (Section 4.2.6.6), coir mesh, or jute fabric (Iowa SUDAS, 2019);

however, coir is most often recommended due to its strength and biodegradable composition (ALSWCC, 2018; NCDEQ, 2013). Ensure the baffle material is durable enough to last the length of the project, as repair as replacement will likely require basin dewatering. Porous baffles should be installed similarly to a silt fence (Section 4.4.10) using posts and wire backing, if necessary. Proper installation requires securing the baffle at the bottom and sides using staples, stakes, trenching, or other anchoring methods to prevent flow from bypassing underneath, over, or around the structure. Staples should be made of 0.125-inch diameter new steel wire formed into a 'U' shape, not less than 12 inches in length, with a throat of one inch in width. Steel posts should be approximately 1-3/8" wide, measured parallel to the fence, and have a minimum weight of 1.25 pounds per linear ft. The posts should be equipped with an anchor plate having a minimum area of 14 square inches. To prevent sagging, a support wire or rope may be tied along the top of the baffle, or use a sawhorse type of support with the legs stabilized with rebar inserted into the basin floor. A nine-gauge high-tension wire is recommended in the former scenario. A porous baffle height of three feet has proven effective, though in some cases, site conditions may warrant taller baffles (ALSWCC, 2018). Baffles should extend the full depth of the basin, including any freeboard and auxiliary spillway depth. Overlap baffles six inches where necessary. Because most sediment settles in the first bay, this area should remain accessible for routine maintenance.


Source: TNWRRC

Research has shown that nearly all porous baffles improve the hydraulic performance of basins through flow dispersion and turbulence reduction (Figure 4.4.12.1.1-A, and as shown by the flow path of the green dye in the above picture) and thereby promote sediment trapping (Farjood et al., 2015; Schussler et al., 2022). Based on the findings of Farjood et al. (2015), the hydraulic performance of basins is significantly influenced by the number, spacing, and mesh characteristics of porous baffles. In the experimental design, the optimal configuration for five baffles included a mesh aperture of one millimeter and 42% open area, evenly spaced throughout the basin. For installations with fewer than four baffles, the best performance was achieved with three baffles having a finer mesh aperture of 0.415 millimeters and 40% open area, placed in the first half of the pond near the inlet. Positioning the first baffle at approximately 17% of the pond length from the inlet enhanced energy dissipation and improved residence time. While finer meshes may improve performance in low-baffle configurations, overly fine meshes combined with high baffle counts can increase

velocity and short-circuiting, ultimately reducing effectiveness (Farjood et al., 2015). Therefore, selecting mesh aperture size and open area should be carefully balanced based on the number and placement of baffles.

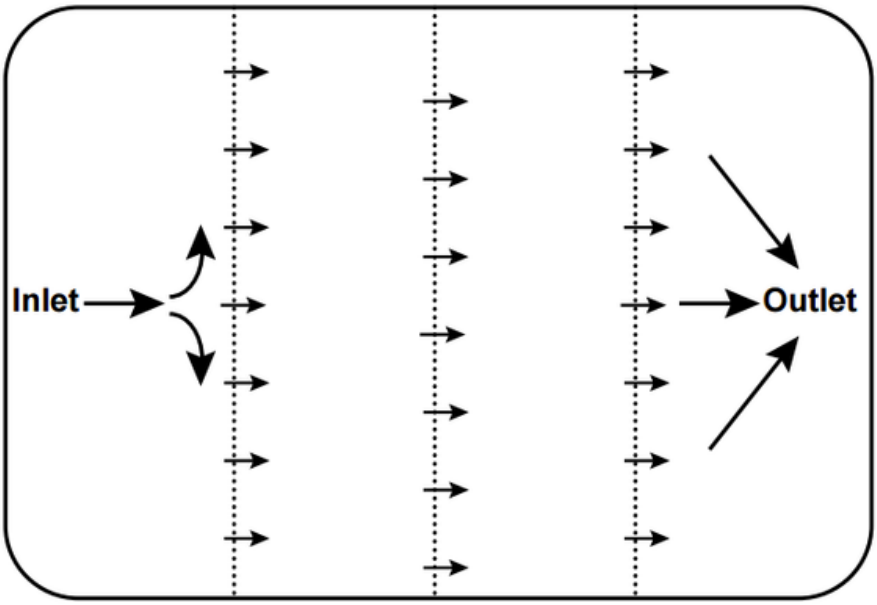


Figure 4.4.12.1.1-A: Flow dispersion and turbulence reduction of porous baffles in a sediment basin. Source: Rivers & McLaughlin (2015).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas*.

Farjood, A., Melville, B. W., & Shamseldin, A. Y. (2015). The effect of different baffles on hydraulic performance of a sediment retention pond. *Ecological Engineering*, *81*, 228-232.

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4.4.12.1.2 Solid Baffles



Source: City of Franklin

Definition and Purpose

Solid baffles are made of non-porous materials that increase the length of the flow path within a sediment basin or trap. Increasing the flow path length increases basin residence time, which promotes the settling of sediment.

Appropriate Applications

Baffles can be installed in any sediment trap or basin. Solid baffles are often used when a sediment basin is on the site, but the available space does not allow for the basin to be constructed with the ideal length-to-width ratio, or the inlet and outlet structures are adjacent to each other, and there is a risk of short-circuiting in the basin.

Limitations and Maintenance

Baffle material must be securely anchored and trenched (unless constructed from plywood) into the ground. Solid baffles may increase the frequency with which accumulated sediment must be removed from the basin, particularly within the first chamber. Inspect for scour along the sides of the baffle and replace baffle material if it is damaged or degraded. Inspect the baffles during rainfall events to confirm that short-circuiting or overtopping of the baffles does not occur.

Planning and Design Considerations

Various types of materials can be used for solid baffles, including plywood, geotextile curtains, floating turbidity curtains (Section 4.4.12.4), or aggregate berms (IDEM, 2021).

Regardless of material, in order for solid baffles to function correctly, they should be set to a height of six inches above the riser crest elevation or spillway elevation when used in a sediment basin or sediment trap, respectively. Most commonly, solid baffles are constructed from four-foot by eight-foot by half-inch-thick plywood. Plywood baffles are to be affixed to four-by-four-inch posts that are set three feet into the ground (KTC, 2015). If geotextiles or floating turbidity curtains are used, they may be staked, stapled, or entrenched into the ground in a similar manner as described in Section 4.4.12.1.1. While various materials for solid baffles may be acceptable, ensure the material is durable enough to last the length of the project. Replacement and repairs of the baffles will likely require dewatering.

Solid baffles can either be used to create flow path openings to create a back-and-forth flow path, as demonstrated in Figure 4.4.12.1.2-A, or the back-and-forth flow path can be established by openings between the baffle and basin embankment, as shown in the section heading picture. When optimally configured, solid baffles divide the basin into multiple cells, enhancing retention time and the settling of sediment. However, solid baffles tend to restrict flow depth and reduce flow area, which increases water velocity. This effect may negate the intended increase in residence time (Farjood et al., 2015). Under specific scenarios and with close inspection, solid baffles can be an effective solution to improving the efficiency of a sediment basin or trap. However, the experimental design from Farjood et al (2015) paired with regulatory guidance from many states suggests that porous baffles (Section 4.4.12.1.1) are typically more effective.

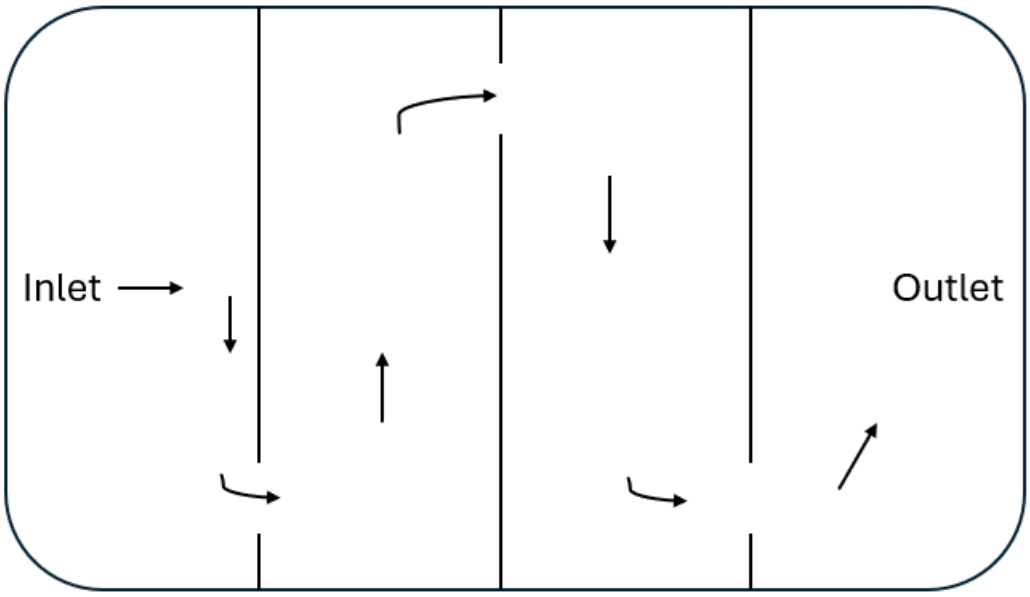


Figure 4.4.12.1.2-A: Modified flow path of a basin with solid baffles. Source: Rich & McLaughlin (2015).

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

Farjood, A., Melville, B. W., & Shamseldin, A. Y. (2015). The effect of different baffles on hydraulic performance of a sediment retention pond. *Ecological Engineering*, *81*, 228-232.

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Rivers, E. & McLaughlin, R. (2015). *Using Baffles to Improve Sediment Basins*. North Carolina State University Extension.

4.4.12.5 Perforated Riser Pipe



Source: City of Franklin

Definition and Purpose

A perforated riser pipe is a dewatering device used in sediment basins (Section 4.4.7). It is a vertical perforated pipe that dewaters the basin at a rate governed by the head of water above the perforations and discharges water to the principal spillway. The perforation (i.e., orifice) with the lowest elevation sets the sediment storage elevation.

Appropriate Applications

Perforated riser pipes are applicable in any sediment basin. They are most ideal when dewatering solely from the top of the water column is not feasible (IDEM, 2021).

Limitations and Maintenance

In order to comply with specific sediment basin detention requirements (i.e., 48 to 72 hours), the diameter and number of orifices within the rise pipe must be computed. This computation is based on the head of water above the orifices which changes over time, thus computation time can be complex and lengthy, may require software, or rely on the interpretation of figures. Further, because a perforated riser pipe does not dewater from the solely from the top of the water surface elevation, sediment removal efficiencies of basins with this dewatering mechanism are lesser than those equipped with a floating skimmer

(Schussler, 2022; Zech et al., 2014). Lastly, orifices have a high potential to become clogged or blocked by trash and debris. Trash guards are highly recommended.

Planning and Design Considerations

Perforated riser pipes are typically made of PVC or metal and are typically connected to the principal spillway outlet. These structures must be stable, durable, and designed to resist flotation and external loading. The riser is anchored in concrete to maintain position and functionality. Multiple perforations, protective mesh, trash guards, or anti-vortex devices may be ideal to prevent clogging (IDEM, 2021).

Riser pipes often have multiple columns of orifices in which the vertical spacing is usually between six inches and one foot. As opposed to a single orifice, perforated risers have many openings, thereby reducing the likelihood of total blockage or failure, and dewatering does not solely occur from the sediment storage elevation. The number of columns, vertical spacing of orifices, and diameter of each orifice all impact the dewatering time and therefore must be selected carefully (Jarrett, 2016). Traditionally, dewatering time was an iterative process governed by the orifice equation:

$$Q = N \times C_d \times A \times (2 \times g \times H)^{1/2} \quad \text{(Eqn 29)}$$

where Q is the outflow rate (cubic feet per second), N is the number of orifices, C_d is the discharge coefficient (dimensionless, typically set at 0.6), A is the cross-sectional area of the orifice (square feet), g is acceleration due to gravity (32.2 feet per square second), and H is the static pressure head (feet, measured from the water surface elevation to the centerline of the orifice). The iterative process begins with assuming an area for the orifice. Iterations must be conducted at sufficiently small differences of H over the orifice centerline, as outflow will change with H. Obtain a corresponding outflow rate for each iteration of H until outflow is negligible. The time between each H iteration would need to be computed via level pool methodologies and summed. If the total time is not within the desired dewatering time, the process must be restarted with a different assumed orifice area.

As this methodology may be time-intensive and detail-oriented, dewatering time through in orifice is more efficiently computed via level pool methodologies in software such as HEC-HMS. Level pool routing in software will be particularly beneficial when riser pipes feature numerous orifices at various elevations. The most time-efficient and simplistic method would be using figures, if they exist for specific conditions. For example, the most common riser configuration is four columns of holes spaced at six-inch vertical intervals (Figure 4.4.12.2.1-A).

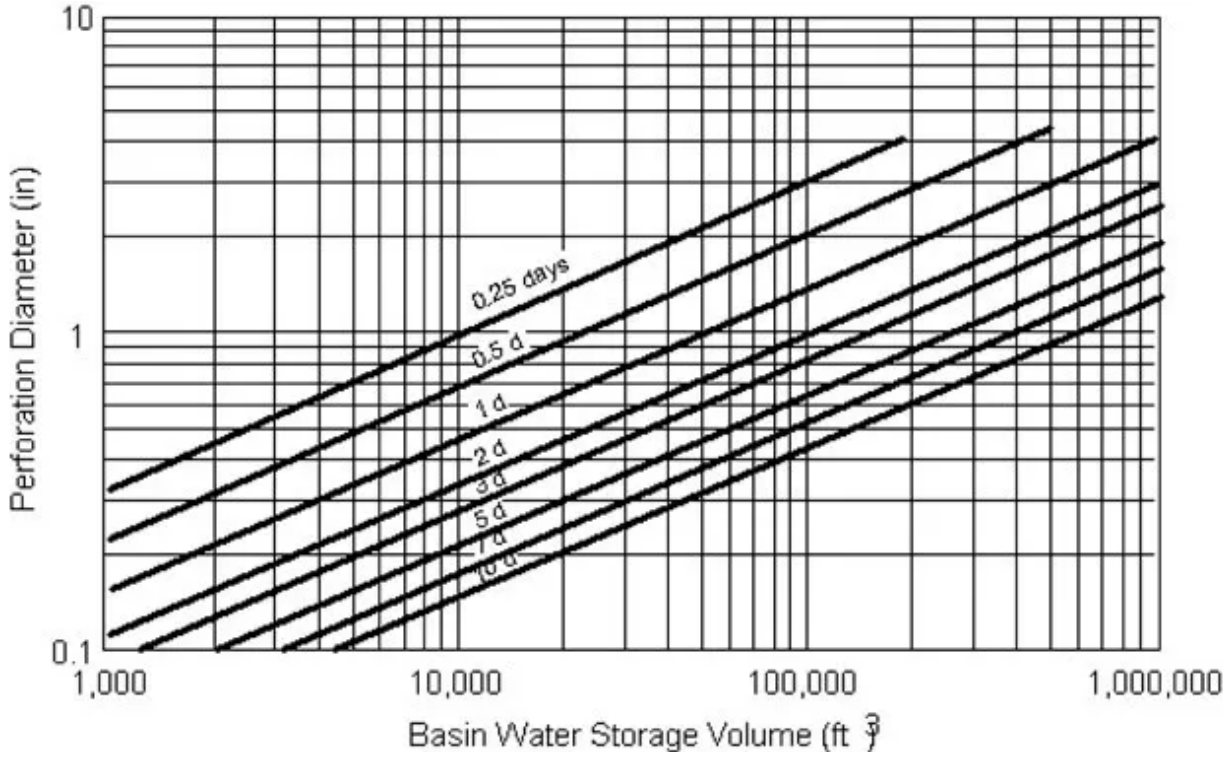


Figure 4.4.12.2.1-A: Design chart for a perforated riser with four columns of orifices spaced at six-inch vertical intervals. Source: Jarrett (2016).

Example Application

Example courtesy of Jarrett (2016)-

Given:

A sediment basin with a live storage zone of 40,000 cubic feet and a required dewatering time of 48 hours.

Determine:

The orifice diameters for a riser pipe.

Solution:

Using a four-column riser, Figure 4.4.12.2.1-A can be used to determine the required hole diameter of 0.67 inches. Therefore, the perforated riser for this basin would be a vertical pipe with four vertical columns of 0.67-inch diameter holes drilled, with the bottom row of holes located at the sediment storage elevation, with additional 0.67-inch diameter holes located on a six-inch interval upwards from the bottom holes.

References

IDEM. (2021). *Indiana Storm Water Quality Manual: Sediment Traps and Basins*.

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Schussler, J. C. (2022). *Improvements in Stormwater Detention Technologies through Large-Scale Testing Techniques*. Auburn University.

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4.4.12.2.2 Skimmer



Source: City of Franklin

Definition and Purpose

A skimmer is a floating device used to dewater a sediment basin (Section 4.4.7) at a controlled rate by discharging water to the principal spillway. Runoff captured in the sediment basin is dewatered by the skimmer from just below the water surface elevation and back to the sediment storage elevation.

Appropriate Applications

Skimmers are applicable in most sediment basins. They are generally used to enhance sediment trapping capabilities compared to perforated riser pipes (Section 4.4.12.2.1).

Limitations and Maintenance

In order to comply with specific sediment basin detention requirements (i.e., 48 to 72 hours), skimmers are selected by the size of the opening (orifice) used to dewater the basin. The orifice size is computed by equations governed by basin dimensions and the flow capacity of the orifice. While these methods are generally reliable for a particular skimmer, manufacturers produce various products that have their own hydraulic characteristics that can impact the dewatering rates (Sharpe et al., 2023; Sprague et al., 2015). Thus, it is important to periodically inspect the dewatering rate of skimmers to ensure it matches the anticipated dewatering rate. Most skimmers come with an adjustable orifice; therefore,

adjusting its size to ensure proper dewatering times is simple, such that there is adequate access to the skimmer.

A few other limitations also exist for skimmer use. Skimmers are likely to clog, whether it be from trash and debris or from freezing conditions (IDEM, 2021). Avoid using skimmers if extended periods of freezing conditions are anticipated. Further, ropes tied to the floating head of the skimmer and extending to the embankment can be a valuable tool to dislodge any debris partially clogging the skimmer without needing to access the basin. The rope may also be useful if needing to adjust the orifice size. Additionally, because the skimmer is a floating device, a stone pad should be constructed below the skimmer (setting the sediment storage height) such that the skimmer does not dewater the entire basin. Lastly, ensure all skimmer components and connections are free of cracks, leaks, and deterioration.

Planning and Design Considerations

Manufactured skimmers are highly encouraged as opposed to contractor-assembled devices. Various manufacturers produce products that are readily available with technical specifications and performance standards, making the process of selecting a skimmer quick and simple. Skimmers typically consist of a flexible boom attached to a perforated inlet and outlet pipe, with pipe diameters typically ranging from one and a half to eight inches. The boom should be free to rise with water levels and should be protected from twisting, kinking, or breaking, especially in cold weather, using restraints like guide bars, rope tethers, or landing devices such as aggregate pads, concrete blocks, or gabion baskets. These landing devices also prevent the skimmer from settling into sediment, blocking inlet perforations (if used), and dewatering the entire basin. The stone pad should be of sufficient size to hold the skimmer and constructed from #57 stone. Further, the pad is to be at a minimum of four inches in thickness, extending to an elevation that corresponds to the elevation of the sediment storage zone. Skimmer orientation should also be considered; directing the boom toward shallow shoreline areas can simplify maintenance and allow for effective landing device placement.


Source: TDEC

To begin the sizing of the skimmer, first note the design volume of water that will be released from the basin in cubic feet (V, i.e., live storage volume) and the desired dewatering time in days (t_d). Then divide the V by t_d to obtain a dewatering flow rate. Next, use manufacturer performance charts to select the appropriate skimmer size and orifice diameter. However, most skimmers come with a plastic plug that can be drilled, forming a hole that will limit the orifice of the skimmer to any desired rate. Thus, for a basin that requires a more specific

dewatering time, an orifice diameter can be calculated via the orifice equation (Eqn 29) as described in Section 4.4.12.2.1.

Because skimmers are floating mechanisms, they dewater the basin from or just below the water surface elevation. This is advantageous because basins retain/capture sediments through the sedimentation process. Thus, as the hydraulic retention time in the basin increases, settling sediments will migrate further down the water column, yielding cleaner water at the top of the column. Because of this theory, skimmers have been labeled as the most effective basin-dewatering mechanism (Zech et al., 2014). In fact, Schussler (2022) found that sediment basins equipped with a floating skimmer retained 12% more sediment by weight than a sediment basin equipped with a perforated riser, an impressive increase considering sediment basins already capture large amounts of sediment within the basin itself [e.g., typical removals of 95% or greater when basins were constructed following TDOT and TDEC recommendations (Emmett, 2022)].

Example Application

Example 1:

-Example from ALSWCC (2018)-

Given:

A sediment basin with a live storage zone of 20,000 cubic feet and a desired dewatering time of 72 hours.

Determine:

An appropriately sized skimmer from manufacturer XYZ.

Solution:

Step 1 – Compute the required outflow rate: Divide the outflow volume by the desired dewatering time.

$$Q = V / t_d = 20{,}000 / 72 = 278 \text{ ft}^3\text{/hour} = 6{,}670 \text{ ft}^3\text{/day}.$$

Step 2 – Use provided specifications from the manufacturer to size the skimmer: Manufacturer XYZ specifies a 2-inch skimmer with no orifice has an outflow rate of 5,429 ft^3/day, and a 4-inch skimmer with a 2.5-inch diameter orifice has an outflow rate of 8,181 ft^3/day. The 4-inch skimmer with a 2.5-inch diameter orifice was selected as the other skimmer does not meet the dewatering rate and therefore would dewater the basin in a time longer than 72 hours.

Step 3 – After upsizing the dewatering rate to match that of a manufactured device, ensure it does not dewater the basin too quickly. Divide the live storage volume by the

dewatering rate of the manufactured device, and ensure the dewatering time is still within 48 to 72 hours.

$$t_d = V / Q = 20{,}000 / 8{,}181 = 2.44 \text{ days} = 58.7 \text{ hours}$$

The actual dewatering time is within the allowable range and therefore, the 4-inch skimmer with 2.5-inch diameter orifice is an acceptable size from XYZ is an acceptable skimmer.

Example 2:

-Example courtesy of ALSWCC (2018)-

Given:

A sediment basin with a live storage zone of 20,000 cubic feet and a 4-inch skimmer from XYZ manufacturing with a desired outflow rate of 6,670 ft^3/day, such that the actual dewatering time is 72 hours.

Determine:

The orifice diameter.

Solution:

Apply the orifice equation (Eqn 29) and convert units where necessary. The head will vary depending upon the manufacturer's design depth of submergence for a given orifice diameter. Assume manufacturer XYZ has a driving head (H) of four inches for a 4-inch skimmer.

$$Q = N \times C_d \times A \times (2 \times g \times H)^{1/2}$$

$$6670 \frac{ft^3}{d} = 1 \times 0.6 \times \frac{\pi \times d^2}{4} \times (2 \times 32.2 \frac{ft}{s^2} \times 4 \text{ in})^{1/2}$$

$$0.0772 \frac{ft^3}{s} = 1 \times 0.6 \times \frac{\pi \times d^2}{4} \times (2 \times 32.2 \frac{ft}{s^2} \times 0.333 \text{ ft})^{1/2}$$

$$d = 0.187 \text{ ft} = 2.24 \text{ inch}$$

Thus, a plastic plug drilled to a diameter of 2.24 inches and placed in the 4-inch plastic skimmer would result in a dewatering time of 72 hours for a live storage volume of 20,000 cubic feet.

References

ALSWCC. (2018). *Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas.*

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Sharpe, C., Perez, M. A., & Donald, W. N. (2023). Evaluation of Surface Skimmer Flow Rates and Size Selection. *Transportation Research Record*, *2677*(9), 199-211.

Sprague, J. E., Sprague, C. J., & Ruzowicz, B. (2015). Evaluating Floating Surface Skimmers. In *2015 IECA Annual Conference*.

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4.4.12.3 Dewatering Structure



Source: TDOT

Definition and Purpose

A dewatering structure is a temporary sediment control device designed to process and treat water that has been pumped or drained from excavated or work areas during construction. Dewatering structures ensure slurry water, stormwater, or other non-stormwater accumulated from construction activities is managed and treated before discharge.

Appropriate Applications

Dewatering structures are applicable in situations where sediment laden water must be removed from construction activities through pumping, such as from excavations, trenches, cofferdams, piles, ditches, or other depressed areas where water accumulates (CalTrans, 2017). They are used to manage both stormwater and non-stormwater, including groundwater and water generated during construction operations, and when stormwater and non-stormwater mix. In these applications, dewatering structures provide an effective alternative to sediment filter bags (Section 4.4.12.6) by allowing water to be collected, settled, and filtered before being discharged off-site. They are most effective when placed on flat or gently sloped areas that infiltrate a large quantity of discharge.

Limitations and Maintenance

Dewatering structures are not to be installed within jurisdictional wetlands or within close proximity to water resources, especially ETWs. Filtering devices must be assessed frequently, often daily during operation, and routinely during site inspections. Accumulated sediment is to be removed once half the capacity of the structure is reached. If sediment buildup reduces function, the structure should be repaired or replaced. Removed sediment must be

stabilized onsite or disposed of at approved locations, and if commingled with other pollutants, handled in accordance with the applicable local regulations. Continuous monitoring is also necessary to ensure discharges do not cause downstream erosion, with additional stabilization measures such as blankets, matting, or filter socks applied where necessary.

Planning and Design Considerations

Because water pumped from construction sites typically contains high sediment loads, structures must be selected and designed to effectively capture sediment without causing downstream erosion, scour, or sedimentation. A dewatering structure may not be necessary if water can be discharged to a flat, well-vegetated, and stabilized area capable of filtering sediment and withstanding discharge velocities. Various types and innovations of dewatering structures (conventional riprap and a geotextile, portable tanks, filter boxes, and a straw bale/silt fence pit) are applicable and presented herein, where the most ideal type is governed by site-specific conditions (VDEQ, 2024). Discharges must remain within project limits whenever possible, with disposal required if constraints or contamination prevent onsite release. Do not discharge polluted waters as indicated by odor, oily sheen, discoloration, or foam (CalTrans, 2017). Offsite stormwater should always be diverted away to maintain efficiency.

While there are various types of innovations for dewatering structures, most can be designed by considering three components: size, location, and lining. The design of dewatering structures must ensure that pumped water passes through the filtering device without overtopping the system. Pumping operations should cease once the water level nears the top of the structure, allowing the basin to drain down naturally; any remaining water may only be discharged after at least six hours of settling. Required storage volume (cubic feet) is calculated by multiplying pump discharge (gallons per minute) by 16 (TDOT). Placement of the structure is also critical; it should be located downhill and away from stockpiles or spoil piles to prevent recontamination. Further, they should not be positioned where treated water could return to the active workspace. To enhance sediment removal, the structure should be placed such that a well-established vegetated area is downgradient. VDEQ (2024) recommends a vegetated filter length of 75 feet while TDOT recommends 30 feet. In cases where an excavated basin is used, such as a straw bale or silt fence pit, filter fabric lining may be applied to minimize scour and prevent soil within the structure from mixing with discharged water. Additional device-specific design requirements are outlined in the following paragraphs.

The conventional riprap and geotextile structure shall consist of class A-1 riprap and a type II geotextile fabric. Stake the geotextile fabric into the ground at a maximum six-foot

intervals. TDOT recommends a maximum height of 30 inches and stone placed at 2H:1V side slopes, requiring adequate space within the right-of-way or, if necessary, a construction easement for installation and maintenance. An optional excavated area at least three feet deep may be incorporated to capture additional sediment, though its volume is not counted toward total storage since water within it may not fully drain between pumping operations. To ensure proper filtration, a silt fence with wire backing is ideally installed across the downslope side of the structure, preventing residual sediment from leaving with the discharged water.

Alternative to a conventional riprap and geotextile structure, a straw bale or silt fence pit may be used. This structure consists of perimeter barriers (straw bales or silt fence), a stone outlet using riprap and aggregate, and a wet storage pit excavated at least three feet below the perimeter base. The face of straw bales is lined with filter fabric, and installation follows standard barrier guidelines. Capacity is measured from the pit floor to the stone weir crest (VDEQ, 2024).

A portable sediment tank is typically constructed from steel drums, sturdy wood, or other strong materials capable of withstanding the water pressure it must hold. The tank must be at least two feet deep and should be placed where sediment cleanout and disposal can be easily managed without disrupting construction. Tanks should be designed with an emergency overflow.

A filter box is built from durable materials such as steel or sturdy wood, often using 55-gallon drums welded together. The bottom is made porous, either by drilling holes or other methods, which are covered with at least 12 inches of coarse aggregate, sometimes reinforced with hardware cloth. Because water passes quickly through the aggregate, the discharge should flow across at least 50 feet of well-vegetated land for further filtering (VDEQ, 2024). The stone filter should be cleaned and replaced if it becomes clogged. Since the filter box allows minimal settling time, it should only be used where other methods are not feasible.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

CalTrans. (2017). *Construction Site Best Management Practices (BMP) Manual.*
TDOT. *Drainage Manual Ch10*.
VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.4.12.4 Floating Turbidity Curtain





Source: TNWRRC

Definition and Purpose

A floating turbidity curtain is an in-stream sediment control measure designed to minimize the spread of suspended sediments from construction activities occurring adjacent to or within a water body. It consists of a geotextile filter fabric or a plastic sheet with holes suspended from floats and held vertically in place by a weighted ballast chain or other anchorage system. Rather than stopping water movement, the curtain isolates the construction zone, allowing sediment to settle before reaching the surrounding watercourse. The primary purpose of a turbidity curtain is to protect water quality by preventing sediment migration into the larger water body.

Appropriate Applications

Floating turbidity curtains are applicable in both watercourses where construction activities result in unavoidable sediment disturbance, such as dredging, filling, or bridge pier installation. They are most effective in shallow waters, typically less than 30 feet deep, where conventional onshore sediment controls are not feasible. Common deployment locations include coastal zones, river channels, and environmentally sensitive areas. Suitability depends on site specific conditions such as hydrodynamics, wave action, and operational constraints like navigation access (Oglivie et al., 2012). Further, they may be used as baffles in sediment basins.

Limitations and Maintenance

Floating turbidity curtains become less effective when used in areas with flow velocities exceeding five feet per second and should never be installed across active stream channels, as strong currents, wave action, and rapid water level changes can submerge or damage the

curtain (TDOT; VDEQ, 2024). Installation errors, such as poor anchoring, gaps between screen sections, or incorrect placement relative to flow, can compromise performance (Oglivie et al., 2012). Repairs should follow manufacturer guidelines, and removal must be done carefully once sediments have settled to avoid resuspension. Curtains should not be left in place once construction activities conclude and soil stabilization is complete. Curtains must also be properly cleaned, repaired, and stored after use to preserve their integrity for future deployments.

Installing turbidity curtains in jurisdictional waters requires additional permits, such as an ARAP, and therefore, both the conditions of the CGP and ARAP must be followed. A Section 404 permit from USACE may also be required, and if the proposed bank protection is to be completed in a TVA reservoir, a TVA 26a permit may also be necessary. Local permitting may be required in addition to state and federal permitting requirements. Consider the criteria and conditions of the necessary permits during the planning stages of the project and EPSC plans.

Planning and Design Considerations

A floating turbidity curtain usually consists of a geotextile filter cloth that is permeable enough to allow water to pass through while retaining sediment and promoting settling; however, other designs that allow water passage and promote settling are also acceptable. The curtain is suspended from buoyant floats and anchored both at the shoreline and within the water to maintain its shape and position. A ballast chain embedded at the lower edge of the curtain helps keep it vertical and provides tensile strength under stress. When used, floating turbidity curtains are typically the last line of defense for preventing sediment transport. As such, potential failure of a turbidity curtain can result in clear violation of the CGP, and therefore, it is imperative that other EPSC measures are installed and functioning properly to limit the amount of sediment received by the curtain.

When selecting a turbidity curtain, it is crucial to assess site specific conditions such as water depth, flow velocity, wave action, and sediment loads (Oglivie et al., 2012). Curtains should never be installed across the main flow of a water body; instead, they should be aligned parallel to the current to prevent structural failure (ALSWCC, 2018; VDEQ, 2024). For river systems, lower velocities are generally found near the shore on straight reaches, whereas higher velocities occur on the outside of bends. Similarly, in tidal or wave-impacted waters, curtains must be carefully designed to account for fluctuating water levels and stresses induced by wave action and/or boat traffic.

Turbidity curtains are generally classified into three types based on environmental conditions. Type I is used in calm, protected waters without current or wave action. Type II is

suited for currents up to 3.5 feet per second and some wind or wave influence. Type III is appropriate for more challenging environments with stronger currents up to 5 feet per second, tidal fluctuations, and significant wave action (ALSWCCC, 2018; VDEQ, 2024). The selection of curtain type should be matched to the hydraulic conditions of the site to avoid structural stress or failure. In areas with strong hydrodynamics, the curtain depth should not exceed 10 to 12 feet to prevent excessive loading and "billowing" of the fabric.

Design considerations also include ensuring an adequate gap, typically 12 inches, between the curtain bottom and the waterbed to prevent burial and resuspension of sediment during removal. In shallow environments less than two feet deep, curtains may be secured with stakes driven into the substrate instead of using floating anchors. Where high sediment containment is critical and full-depth coverage is necessary under tidal conditions, either woven pervious filter fabric or impermeable material (with holes) can be used, but whichever reduces the hydraulic pressure on the barrier while maintaining filtration effectiveness is preferred.

Curtain segments are generally fabricated in 50 to 100-foot sections, joined with secure connections that maintain full material strength. The number of joints should be minimized, and a minimum of one anchor is recommended at each joint, with additional anchoring based on wave and flow conditions. For safety and visibility, curtains should be a bright color, typically yellow or international orange, and may require lighted buoys in navigable waterways, subject to agency requirements like those of the USACE or TVA.

Finally, to enhance effectiveness in highly active water bodies, dual parallel curtains separated by rigid barriers (e.g., wooden boards or pipes) can be deployed. This configuration helps reduce sediment escape and improves overall containment. Throughout the design and installation process, adherence to manufacturer specifications and regulatory guidelines is essential to ensure both environmental protection and operational compliance.

The installation and removal of turbidity curtains must be carefully planned and executed based on the waterbody conditions. In calm waters such as lakes or ponds (Type I installations), installation typically involves placing end stakes or anchor points, using anchor buoys if bottom anchors are required, then towing the furled curtain into place and securing it. Once aligned, additional anchors may be added to maintain the curtain position, and the furling lines are cut to release the skirt into the water. External anchors such as fluke, plow, or mushroom types must be placed to ensure the curtain remains in a stable vertical position, with sufficient slack in anchor lines to accommodate tidal or wave-induced

movement. In moving water environments (Type II and III installations), all anchor points should be securely installed prior to curtain deployment. Anchor buoys are essential in these conditions to prevent flotation submersion. The curtain should be secured starting from the upstream end and proceeding downstream, with its lay assessed before unfurling. Importantly, all anchor lines must be attached to the flotation line, not the curtain bottom, to avoid structural failure. In cases where silt buildup in mid-channel poses a risk, curtains may be installed in a sharp, long-sided "V" configuration to deflect cleaner water and contain sediment-laden flows near the worksite. Removal of the curtain must be done with care to avoid disturbing settled sediment, ensuring protection of water quality during final demobilization.


Source: City of Franklin

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control, and Stormwater Management on Construction Sites and Urban Areas.*

Oglivie, J., Middlemiss, D., Lee, M., Crossouard, N., & Feates, N. (2012). Silt curtains-a review of their role in dredging projects.

TDOT. *Drainage Manual Ch10*.

VDEQ. (2024). *Virginia Stormwater Management Handbook.*

4.4.12.5 Flocculants and Polymers



Source: ALSWCC (2018)

Definition and Purpose

Flocculants and polymers are chemical treatments used on construction sites to promote the aggregation of fine suspended particles, such as clays and silts, into larger masses known as "flocs." These flocs settle more rapidly than individual particles, improving the clarity of stormwater runoff before it leaves the site. These chemicals may be derived from natural or synthetic polymers and can also be applied to soil surfaces or erosion control materials to stabilize sediment and limit erosion (USEPA, 2021).

Appropriate Applications

Flocculants and polymers are most applicable on construction sites where conventional EPSC measures alone cannot effectively retain sediments onsite. Therefore, chemical treatment is to be used in conjunction with other EPSC practices. They are particularly suited for situations requiring stringent turbidity control when discharging to sensitive water bodies, such as lakes, ponds, and trout streams, or in areas where soil types produce particles too small and buoyant for settling or filtration (ALSWCC, 2018; TDOT).

Flocculants can be applied in granular powder or liquid form on graded or disturbed areas, causing soil particles to bind together and become more resistant to wind erosion and raindrop detachment. Further, they can be applied in solid form, as a floc log, or gel in areas of concentrated flow, such as swales or channels, or within structures such as sediment basins, traps, sediment filter bags, etc. When applied as the latter method, the chemical dissolves as water comes in contact with it, binds suspended sediments together, and increases the rate of sedimentation. Kazaz et al. (2023) observed agitation and mixing

improvements up to 96% when flocculants were applied in concentrated flow scenarios. Selection of the appropriate chemical is site specific, requiring evaluation of soil characteristics and regulatory approval to ensure effectiveness and avoid aquatic toxicity.

Limitations and Maintenance

Flocculants and polymers require routine inspection and careful handling to ensure effectiveness and compliance with regulations. Specifically, any form of chemical treatment requires approval from TDEC-DWR before application. Dosing systems should be monitored daily, and solid flocculant products, such as floc logs, inspected at least twice weekly or after significant rainfall to confirm proper placement, moisture, and exposure. Only approved formulations, such as anionic, linear polyacrylamide (PAM) meeting EPA and FDA purity standards, are permitted in certain jurisdictions, and applications must avoid direct discharge to streams, wetlands, or other water resources, with minimum setback distances maintained to protect sensitive areas. Cationic forms of flocculants have a positive charge that is extremely toxic to aquatic species and are not permitted for use in Tennessee.

Planning and Design Considerations

Various products may be applicable for use as a flocculant; however, products such as with PAM or chitosan have been found to be the most effective (Kazaz, et al., 2023). The selection of a flocculant should be based on site-specific soil characteristics, often determined through testing methods such as a “jar test,” to ensure compatibility and maximize performance. Only products meeting regulatory purity and safety standards, such as anionic PAM with less than 0.05% free acrylamide, should be used, and they may require approval from localities. In areas where flocculants are permitted, applications must avoid direct discharge into sensitive water resources and maintain required buffer distances.


Source: TDEC

Proper mixing, distribution, and contact time are critical for effective flocculation, with passive mixing via channel turbulence often preferred. Flocculants dissolve slowly, requiring adequate agitation and time to become fully active, and must be reapplied after significant runoff events as they bind to sediment particles. Application rates should follow manufacturer guidelines and consider site conditions such as slope gradient (steeper slopes require higher rates but yield diminished benefits without strong vegetative cover). Certain soil types, such as sandy soils with low clay content, may see little to no improvement from flocculant use. Practical planning should also account for cost, material handling hazards,

and ensure that all treated runoff passes through a downstream sediment control feature before leaving the site.

When incorporating PAM as a sediment control measure, its application must be planned, implemented, and monitored in accordance with manufacturer specifications, regulatory requirements, and site-specific conditions. The following design considerations apply (ALSWCC, 2018; MPCA, 2023; NCDEQ, 2013; TDOT):

General Requirements

- Apply PAM only after all appropriate EPSC measures have been implemented.
- Use anionic PAM products only; cationic forms are prohibited due to aquatic toxicity.
- PAM products must meet EPA and FDA acrylamide monomer limits (≥0.05%), have a density of 10–55% by weight, a molecular weight of 16–24 Mg/mole, be non-combustible, and contain only manufacturer-recommended additives.
- Application must comply with all OSHA safety requirements, Material Safety Data Sheet provisions, and federal, state, and local regulations.
- Vendors should supply, if requested, written toxicity reports verifying that the product meets state and federal water quality standards.

Application

- PAM shall not be applied directly to streams, wetlands, or other surface waters, nor within 30 feet of such resources.
- Treated areas should drain to a sediment basin, trap, etc., for final settling prior to discharge.
- PAM may not be applied downgradient from a sediment basin or trap.

Mixing and Dissolution

- Adequate mixing is essential for effectiveness. Introduce PAM at points of high velocity or turbulence (e.g., slope drains, storm drain junction boxes, or other high-energy flow areas) to maximize contact with suspended sediment.
- Ensure at least 60 seconds of mixing before water enters a sediment basin or trap.
- PAM is water-soluble but dissolves slowly; sufficient agitation and time are required.

Product Forms and Application Methods

- Liquid Emulsions: Apply only in accordance with laboratory-tested rates for the site; ensure uniform coverage and avoid over-application.
- Dry Powder: May be applied via hand or mechanical spreader; mixing with dry silica sand improves distribution. Pre-mixing with fertilizer, seed, or soil amendments is permissible.
- Solid Blocks/Logs: Install upgradient of sediment basins or traps in areas with constant flow. Placement should follow site-specific testing. Protect logs from dehydration by installing them in shaded or enclosed areas.

Limitations of Solid Forms

- PAM release rates from logs are not adjustable and may be inconsistent.
- Wet PAM can collect sediment and debris, reducing effectiveness; dry PAM logs release less material until fully moistened again.
- To maintain performance, reduce upstream sediment loads, and ensure continuous water contact with the product.

Performance and Reapplication

- PAM effectiveness decreases after storm events; reapply after approximately two rainfall events.
- Soil testing is required to match the PAM formulation to the soil type of the site.
- Follow manufacturer application rates to avoid aquatic toxicity from over-application.

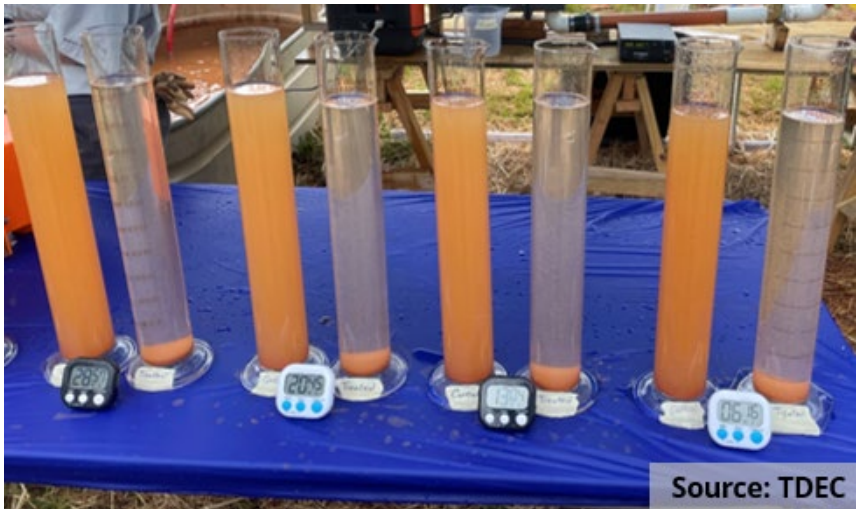
Post-Application Treatment

- After mixing, runoff should be detained in a sediment trap or basin (or other) to allow flocculated particles to settle.
- Stabilize basin inlets with geotextile or stone to prevent gully erosion.
- Consider installing baffles to improve settling efficiency and use surface skimmers for dewatering.

Application Suitability

- PAM is particularly effective in reducing turbidity from high suspended solid loads such as those from dewatering operations, borrow pits, excavation discharges, or stormwater from disturbed areas.
- An application system should be in place to minimize malfunction risk and prevent harm to sensitive water resources, especially those designated as ETWs.

Electrical flocculation, also known as electrocoagulation, is an advanced water treatment process that uses an applied electrical current to destabilize and aggregate suspended solids, colloids, and other contaminants without the need for conventional chemical coagulants. In this process, metal electrodes, typically aluminum or iron, release ions into the water under a direct current, which neutralize particle surface charges and promote floc formation through coagulation and flotation mechanisms.



Compared to traditional polymer- or alum-based flocculation, electrical flocculation offers several advantages: it reduces or eliminates chemical dosing, produces less sludge, and allows for precise control over treatment intensity by adjusting electrical parameters. Studies, including Liu & Perez (2022), have demonstrated that when coupled with physical clarification devices such as lamella settlers, electrocoagulation significantly enhances turbidity removal efficiency, improves settling rates, and shortens treatment times. The technology also provides operational resilience under variable flow and contaminant loads, making it well-suited for construction site runoff treatment where sediment characteristics and flow volumes can fluctuate rapidly. However, considerations such as energy requirements, electrode maintenance, and cost must be evaluated before large-scale implementation.

Example Application

No formal design or quantities are required for this measure and therefore are not presented here.

References

ALSWCC. (2018). *Erosion Control, Sediment Control, and Stormwater Management on Construction Sites and Urban Areas.*

Kazaz, B., Clampitt, J., Perez, M. A., Donald, W. N., Fang, X., & Shaw, J. N. (2023). *Best Practices for Construction Site Stormwater Treatment Using Flocculants* (No. 931-017). Auburn University. Highway Research Center.

Liu, L., & Perez, M. A. (2022). Development and evaluation of lamella settlers combined with electrocoagulation for treating suspended sediment. *Journal of Irrigation and Drainage Engineering*, *148*(6), 04022013.

MPCA. (2023). *Minnesota Stormwater Manual: Construction Stormwater Treatment – dewatering, including chemical treatment and sediment filtration.*

NCDEQ. (2013). *Erosion and Sediment Control Planning and Design Manual*.

TDOT. *Drainage Manual Ch10*.

USEPA. (2021). *Stormwater Best Management Practice: Treatment Chemicals for Particulate Removal from Construction Stormwater.*

4.4.12.6 Sediment Filter Bag





Source: TDOT

Definition and Purpose

A sediment filter bag is a temporary sediment control device made from durable, UV-resistant, non-woven, geotextile fabric designed to retain sediment while allowing water to pass through. These enclosed bags are used primarily during dewatering operations, where sediment-laden water is pumped into the bag, filtering out larger, coarser soil particles before the water is released.

Appropriate Applications

A sediment filter bag may be used whenever sediment-laden water is removed from an area by means of pumping. It should be used in conjunction with dewatering operations for coffer dams, trenches, enclosed ditches, and other construction activities that require the removal of sediment-laden water. It is an alternative for dewatering structures (TDOT).

Limitations and Maintenance

A sediment filter bag should not be placed near or within natural water resources, outlets, inlets, or other sensitive areas. The bag should be placed on flat or gently sloped (and ideally vegetated) areas, so that the outflow cannot concentrate, gain velocity, and serve as a potential source of erosion. Sediment filter bags are not suitable for treating water contaminated with pollutants other than sediment (DEM, 2021). Although effective at capturing sand and other coarse particles, finer materials such as silt and clay may pass through (IECA, 2024), thus, it is common to use in conjunction with flocculants (Section

4.4.12.5) and/or to direct the outflow through an additional measure such as a silt fence with wire backing (Section 4.4.10), wattles (Section 4.4.9), a sediment basin (Section 4.4.7), etc.

Sediment filter bags should be removed and disposed of once they have reached their capacity and have been dried out. Removal should only occur during dry conditions. A full sediment bag can weigh as much as seven tons; therefore, the designer should ensure that there will be adequate access for necessary equipment to lift and remove the bag (TDOT). Bags are prone to rips and tears during removal; use caution when handling the bags. To limit instances of rips and tears, bags can be placed in containers or on pallets prior to use so that moving equipment will not directly contact the bag. If a container is used, the container should be clean so that it will not be a source of other contaminants or pollutants.

Planning and Design Considerations

Designing or selecting the appropriate sediment filter bag requires consideration of several key factors to ensure effective sediment removal and system reliability.

The bag should be sized based on the anticipated flow rate, total water volume, and sediment particle size expected in the discharge. Standard bag sizes of 15 by 10 feet and 15 by 15 feet have maximum flow rate capacities of 1,500 and 2,000 gallons per minute, respectively. Undersized bags increase the risk of failure through bursting, seam rupture, or hose detachment. Follow all manufacturer specifications when sizing bags, particularly in relation to pump output and site-specific soil conditions.

Filter bag material is also critical; nonwoven, polyethylene, or geotextile fabric should provide adequate permeability to capture target sediment sizes while allowing water to discharge efficiently. Minimum filter bag properties are provided in Table 4.4.12.6-A. In areas with high clay content, larger bags or supplementary measures such as flocculants may be needed to prevent rapid clogging.

Placement is equally important; sediment filter bags should be located in areas where discharge can be managed without causing erosion and where the bag can be safely accessed for monitoring, maintenance, or removal. Secure inlet and outlet connections are essential to prevent bypassing or dislodgment during operation.

Table 4.4.12.6-A: Sediment filter bag material specifications. Source: TDOT.

Physical Property	Test	Value
Weight (oz/yd)	ASTM D 3776	10
Tensile Strength (lbs)	ASTM D 4632	250
Tensile Elongation at Break (%)	ASTM D 4632	50
Puncture Strength (lbs)	ASTM D 4833	115
Trapezoidal Tear (lbs)	ASTM D 4533	100
Mullen Burst (lbs)	ASTM D 3786	350
Water, Flow Rate (gpm/ft^2)	ASTM D 4491	80
Permittivity (sec^{-1})	ASTM D 4491	1.2
UV Resistance (%)	ASTM D 4355	70

Installation of sediment filter bags should be on level, stable ground free of sharp objects or debris that could puncture the bag. Ideally, the bag is placed on a pad of #57 stone, six inches in depth. This pad should be wrapped in geotextile fabric (Type III) and extend beyond the footprint of the bag to provide stability and promote even drainage. Filter bags should not be placed directly on slopes; if unavoidable, an aggregate pad may be used to level the site. To prevent erosion and secondary sediment transport, bags should not be located in concave areas, concentrated flow paths, or in or near water bodies. The pump discharge hose should be securely connected to the bag using a tight, leak-free connection, preferably with steel hose clamps. Outflows should be directed to a stable, protected discharge path; energy dissipation measures may be necessary depending on the pump flow rate. Though not ideal, a sediment filter bag may be applied on pavement or other impervious surfaces. In such situations, the bag should be placed on a minimum six-inch layer of mineral aggregate (size 57) with geotextile fabric. In addition, it should be surrounded by filter socks, wattles (Section 4.4.9), or other appropriate measures.


Source: TDOT

A sediment filter bag is considered full once the accumulated sediments have reached a depth of six inches. This corresponds to approximately four cubic yards of material, which can weigh as much as seven tons. Once the bag has reached this point, it should be removed and replaced with a new one. Bag disposal should be in a landfill or other suitable area. It is not acceptable to open the bag and spread the collected sediments on site. For ease of maintenance and disposal, smaller filter bags may be placed on pallets, on trailers, or within containers. Otherwise, lifting and removal of the bag may be facilitated with lifting straps.

The designer should allow sufficient room and a clear path to allow access to the equipment needed for bag removal.

Example Application

No formal design or quantities are required for this measure and, therefore, are not presented here.

References

IECA (2024). *Sediment Filter Bag: A Summary of IECA Design Standard*.
IDEM. (2021). *Indiana Storm Water Quality Manual: Sediment Traps and Basins*.
TDOT. *Drainage Manual Ch10*.

4.5 Multipurposed Measures

Understanding site specific conditions and drainage patterns will assist the designer in selecting which measure to use at various locations throughout the site. Certain practices are best suited for specific hydrologic conditions such as dissipating raindrop impact, managing sheet flow, shallow concentrated flow, or open channel flow, while others may be more broadly applicable. Selecting an unsuitable measure increases the risk of failure, potentially resulting in sediment discharges and violating the CGP. For example, a common design error involves placing silt fence along the entire site perimeter without considering concentrated flow paths or topography. When exposed to concentrated flows, silt fences are prone to failure and would not be an appropriate measure. Moreover, installing silt fence along the upslope side of a site is generally unnecessary, as it would only intercept run-on from adjacent properties rather than treating onsite runoff. In some cases, this may inadvertently obstruct stormwater and cause localized flooding. A more effective strategy would be to install a diversion to reroute run-on around the site. Additionally, the materials used in constructing a measure influences their applicability. For example, a Type I RECP (Section 4.2.6.6) may only be pertinent on a hillside, while Type IV may be applicable on the hillside or in areas of shallow concentrated flow.

In many cases, multiple EPSC measures may be equally appropriate for a given condition (Table 4-A). When this occurs, consider factors such as cost, material availability, reusability during different phases of construction, logistical considerations, and environmental impacts. From a regulatory perspective, these factors are not of concern, provided that the selected measure is appropriate for the intended treatment type and is implemented effectively. For example, both wattles and silt fences are commonly used to manage sheet flow. Substituting one for the other is generally acceptable to regulators; however, site specific conditions, such as shallow bedrock that prevents proper silt fence installation, may necessitate the use of wattles to ensure functionality and compliance. Further, wattles with a biodegradable netting would pose less to no environmental impact compared to silt fence. As another example, wattles may be used as barriers to treat sheet flow or as check dams within ditches to slow runoff and promote sedimentation of turbid waters. The effectiveness of such measures depends not only on the selection of an appropriate product but also on its proper application and installation. EPSC measures are versatile tools, and it is ultimately the responsibility of the engineer to determine the most effective measure for each location, as well as the correct method of installation.

Table 4-A: EPSC measures and their primary (bolded font) and secondary purposes (normal font).

EPSC Measure	Erosion Prevention	Runoff Velocity Control	Sediment Control
Diversion Berm or Dike	**X**		
Diversion Channels	**X**		
Diversion Pipes and Culverts	**X**		
Grade Control	**X**	X	
Slope Drain	**X**		
Channel and Swale Stabilization	**X**	X	
Compost Blanket	**X**		
Gravel Access Road Stabilization	**X**		
Hard Armoring	**X**	X	
Hydroseeding, Soil Binders, and Tackifiers	**X**		
Rolled Erosion Control Products	**X**	X	
Sod	**X**	X	
Stabilization with Plastic Covering: Emergency	**X**		
Stabilization with Straw or Mulch: Temporary	**X**		
Topsoil Stockpiling	**X**		
Vegetation and Landscaping: Permanent	**X**	**X**	
Vegetation: Temporary	**X**	**X**	
Benching and Terracing		**X**	
Channels and Swales		**X**	
Check Dams, Enhanced, Gabion		**X**	X
Energy Dissipator		**X**	
Level Spreader		**X**	X
Riprap Apron		**X**	X
Stilling Basin		**X**	X
Surface Roughening		**X**	X
Brush Fabric Barrier		X	**X**
Filter Berms: Slash Mulch		X	**X**
Filter Strip		X	**X**
Inlet Protection: Area			**X**
Catch Basin Filter Assemblies			**X**
Inlet Protection: Curb			**X**
Inlet Protection: Filter Rings for Culverts		X	**X**
Sediment Basin		X	**X**

Sediment Trap	X	**X**
Sediment Tubes and Socks	X	**X**
Silt Fence and Associated Variants	X	**X**
Weep Berms	X	**X**
Baffles: Porous, Solid	X	**X**
Perforated Riser Pipe, Skimmer	X	**X**
Dewatering Structure	X	**X**
Floating Turbidity Curtain		**X**
Flocculants and Polymers		**X**
Sediment Filter Bag		**X**

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Whitman, J. B., Schussler, J. C., Perez, M. A. & MAPLE Consulting LLC. (2025). *Use of Sustainable Materials for Erosion and Sediment Control Practices: A Synthesis of Highway Practice*. National Cooperative Highway Research Program Synthesis 643.

Chapter 5 – Equivalency and Treatment Trains

A treatment train system combines multiple EPSC measures in a sequence that manages stormwater and soil movement throughout a construction site. The goal is to control erosion at the source, reduce runoff velocity, and capture transported sediment before it leaves the disturbed area. The first and most important step in erosion prevention and sediment control is minimizing soil disturbance and preserving the existing vegetative cover. Well-planned projects avoid and minimize adverse impacts, and where disturbance is necessary, the least impactful construction processes are chosen. Final stabilization should be implemented as soon as possible.

The treatment train begins with erosion prevention measures such as minimizing soil disturbance, protecting riparian buffers and other protected areas, and combines erosion protection such as soil stabilization, slope protection, diversion ditches, and temporary seeding to minimize soil detachment. Runoff is then directed through intermediate controls such as check dams or filter berms to slow flow and promote settling. At the downstream end, final sediment controls (e.g., silt fences, sediment basins, or outlet protection) provide additional treatment before discharge. Properly designed and maintained treatment train systems ensure that each control practice supports the next, providing layered protection against sediment loss and maintaining compliance with permit requirements.

When site constraints preclude the implementation of typical EPSC measures, treatment trains that achieve the same (or better) expected result in terms of sediment retention (i.e., equivalency) can be used instead of a single EPSC, such as a sediment basin, a sediment trap, or a riparian buffer. For example, a sediment basin may not be practicable when the site has limited or dispersed available space, a low depth to groundwater table, limited access to the ideal sediment basin location, presence of bedrock near the ground surface, permanent detention is not planned, etc. Therefore, the design engineer may consider a treatment train system that, when acting collectively, achieves the same, or better, expected results as the single structural measure. Similarly, if construction activities result in encroachment of and disturbance to a stream buffer, an equivalent measure, or treatment train, must be implemented upgradient, documented in the EPSC plans, and provide the same, or better, expected benefits as the stream buffer.

When considering equivalency in EPSC and site design, the engineer is encouraged to consider and/or utilize various EPSC measures and differing designs. In the same regard, structural measures may be required onsite, and while it may be feasible to incorporate them into the site layout, they may not provide the preferred environmental benefit. In such

cases, the engineer is also encouraged to exercise their professional judgement and consider a variety of treatment train options in comparison to the required measure. In either scenario, the engineer is to justify their proposed equivalent design by providing engineering calculations demonstrating efficacy and equivalency to what is required.

5.1 Justification and Calculations

Most structural measures, such as a sediment basin and trap, have design guidance in place to trap sediment generated and transported from the design storm. Engineers use the design storm to determine the sizing of structural measures; however, through equivalency, the required sizing can be distributed amongst a number of sediment control measures within the same drainage area. This can be either a combination of different measures or multiple of the same.

Under this approach, the sizing and treatment of each measure is to be calculated and summed to treat the same or greater volume of sediment. The engineer can implement erosion prevention measures to reduce the sediment yield (Section 2.2) from the site. Specifically, the engineer is to provide calculations demonstrating that the treatment train provides the required treatment volume from the design storm per the CGP, provide calculations demonstrating that a reduced sediment yield (e.g., Eqn 11, Section 2.2.1) will result in the same or better expected benefits of sediment retention, or a combination of the two. Ultimately, this can be achieved by incorporating measures to reduce the required treatment volume if applicable or necessary (e.g., diversions), comparing the volume of runoff treated onsite from the treatment train to that which is required, and reducing the C, LS, or P factors in Eqn 11 (or similar if a different sediment yield estimation method is used).

5.2 Redundancy

Treatment trains are often desirable, as they can be used to capitalize on the varied treatment mechanisms of different EPSC measures. For example, an EPSC measure that relies on sedimentation (such as a sediment basin) can be coupled with an EPSC measure that relies on filtration (such as a filter berm) to target different sediment sizes. Such configurations allow creativity for the engineer and treatment of sediment closer to its source. Additionally, if two EPSC measures of the same type are in series, it may be easier to fit the required treatment volume on the site without the need for excessive grading. That is, multiple smaller measures can be used instead of having to provide space for one large measure. Furthermore, the forebay of a sediment basin may consist of a channel with check dams that widens into the basin or be disconnected from the main basin itself, providing pre-treatment, or additional treatment, for a basin with easier access for maintenance.

5.3 Treatment Train Examples

The engineer may show equivalency from any combination of EPSC measures installed as a treatment train. However, commonly installed treatment trains throughout the state of Tennessee include:

- A series of check dams that can lead to a sediment trap;
- Check dams and sediment traps sequentially in series (linear sites);
- Expedited slope stabilization measures leading to a silt fence or a slash mulch berm;
- Slope drains discharging to a conventional riprap apron, sediment basin, or stilling basin;
- Sheet flow being treated by a series slash mulch berm and/or silt fence;
- Check dams used as a forebay for a sediment basin; and
- Wattles on the slopes breaking up concentrated flow that are placed upgradient of silt fence.

Chapter 6 - Problem Solving and Innovative Best Management Practices

This section provides users with examples of how engineers can be creative in their design and implementation of EPSC measures. The guidelines listed in this handbook are to help the engineer realize best design practices of EPSC measures as well as detailing best installation practices in the SWPPP narratives. Ultimately, the engineer can design a site at their discretion, and their designs may vary from the recommendations herein; however, it is their responsibility for creating a design that keeps the construction site in compliance with the CGP.

6.1 Common Issues and Site Specific Design

SWPPPs require site specific tailoring to function effectively. The performance of EPSC measures depends on how well they are matched to site constraints and hydrology. For example, silt fence loses effectiveness when installed along the perimeter of a site that includes ditches or within drainage ditches where concentrated flow occurs. In the later scenario, check dams or other flow dissipating practices are more appropriate. Similarly, silt fence placed upgradient of the disturbance contributes no treatment benefit and only adds unnecessary cost. Careful evaluation of drainage patterns, disturbance limits, and runoff sources during the design phase enables EPSC measures to be both efficient and cost-effective. Ideally, these considerations are to be used in tandem with implementing measures that have the least environmental impact. For instance, the negative impacts of blasting to create a sediment basin in rocky terrain may offset the benefits of that sediment basin throughout its operational lifespan. Similarly, purchasing more right-of-way land on a linear project to construct a sediment trap is likely more of an economic burden than purchasing and installing bypass pipes that allow unpolluted water to bypass the construction site. This changes the outlook of sediment control to include other aspects such as managing the work, water, and erosion.

6.2 Traditional Erosion and Sediment Control Management

Historically, construction stormwater management has focused on treating the visible symptoms of stormwater impacts including sediment accumulation in receiving waters, wetlands, and on adjacent properties. Although sediment control remains a regulatory priority, the term “erosion control” has often been applied loosely, giving the impression that the causes of sediment transport were not being addressed when management efforts were largely reactive. Over time, stormwater practices have advanced by incorporating soil conservation principles established in agriculture, where controlling soil detachment at its source is fundamental (Edwards et al., 2015; Kazaz et al., 2022). These lessons have shifted construction stormwater management toward a more preventative or proactive model.

Source-control practices such as surface stabilization, vegetative buffers, and velocity reduction now play a greater role in construction stormwater management plans. By limiting soil detachment and exposure, these practices reduce the load on downstream sediment controls and improve overall treatment efficiency. This combined strategy of addressing erosion at its source while maintaining sediment capture downstream represents a more technically sound and science-based approach to construction stormwater management that overall reduce downstream water quality impairments associated with construction.

6.3 Five Pillars of Construction Stormwater Management

Construction stormwater management is essential for minimizing the environmental impacts of soil disturbance. The Five Pillars of Construction Stormwater Management (Fagan, 2020) provide a prioritized framework based on effectiveness and cost efficiency:

1. Manage communication;
2. Manage work;
3. Manage water;
4. Manage erosion; and
5. Manage sediment

This framework guides planning, design, construction, and troubleshooting across a wide range of projects, including transportation, residential, commercial, and utility-scale solar development. Applied effectively, the Five Pillars reduce erosion, control sediment transport, and support compliance with regulatory requirements, resulting in more efficient and environmentally protective construction practices.

6.3.1 Manage Communication

The first pillar, *managing communication*, establishes the foundation for effective stormwater management by aligning project teams, contractors, inspectors, and regulatory agencies on objectives, methods, and performance expectations. Clear communication reduces misinterpretation, minimizes compliance risk, and facilitates efficient implementation of EPSC measures. Key practices in this pillar include delineating sensitive areas through physical demarcation, delivering or obtaining essential trainings and credentialing, conducting routine inspections, allocating resources for maintenance, and engaging stakeholders through outreach.

Communication also directly supports regulatory compliance. For instance, when designers convey that silt fence typically functions for only six months, contractors can incorporate replacement costs into bids, and owners can anticipate additional material purchases if the project extends in duration.

6.3.2 Manage Work

The second pillar, *managing work*, addresses the execution of construction activities with the objective of minimizing soil disturbance, preserving site stability, and protection of natural features. Core practices include phasing and sequencing to limit exposed soil, selective clearing to retain stabilizing vegetation, stabilized access roads that prevent sediment tracking, effective topsoil management to support final stabilization, implementation of water conservation practices, and installation of temporary stream crossings or vegetated filter strips.

For example, specifying on plan sheets that work within a buffer zone is to occur only during dry conditions, followed by immediate stabilization, significantly reduces the risk of sediment discharge. Such measures align construction sequencing with weather conditions, lowering the likelihood of sediment mobilization and ensuring compliance with regulatory requirements.

6.3.3 Manage Water

The third pillar, *managing water*, focuses on controlling runoff to prevent contact with disturbed soils and to reduce erosion risk. Effective water management lessens reliance on downgradient sediment controls by intercepting, diverting, or safely conveying flows before they reach vulnerable areas. Key practices include diversion channels, slope drains, cofferdams, piling, shoring, berms, and other flow control structures (see Section 4.3). On sites adjacent to streams or wetlands, temporary diversions or cofferdams isolate work zones and limit sediment transport. Slope drains and flumes convey water safely down exposed slopes, reducing gully formation and soil loss.

For example, during roadway construction near a wetland, a geotextile-lined diversion channel can redirect stormwater away from disturbed areas, while slope drains discharge runoff into collection points stabilized with conventional riprap. These strategies minimize soil detachment, reduce sediment laden flows, and protect downstream aquatic resources and habitats.

6.3.4 Manage Erosion

The fourth pillar, *managing erosion*, is often viewed as the first line of defense for preventing soil loss on construction sites, though these pillars demonstrate other construction related aspects that come before *managing erosion*. This pillar emphasizes stabilizing exposed soils, controlling runoff energy, and reducing erosive processes such as raindrop, sheet, and rill erosion. EPSC measures include surface stabilization, hydraulic mulching, vegetative

establishment, temporary slope protection, streambank stabilization, hard armoring, etc. (see Section 4.2).

For example, on hillside residential projects, hydraulic mulch can be applied immediately after grading to limit raindrop and sheet erosion. ECBs, preferably those that are environmentally- and wildlife- friendly, on steep slopes hold soils in place until vegetation establishes. In stream restoration, live staking with native cuttings can be paired with log structures to stabilize banks and accelerate recovery (Section 3.3.2.2). At stormwater outfalls, riprap apron and stilling basins dissipate flow energy and prevent scouring, ensuring long-term stability of receiving channels.

6.3.5 Manage Sediment

The fifth pillar, *managing sediment*, serves as the last line of defense in preventing sediment from leaving a construction site. Sediment controls are designed to intercept, capture, and retain mobilized sediment, particularly during storm events, thereby reducing loads to receiving waters. Key practices include perimeter controls such as silt fence and wattles, inlet protection for curb and area drains, track-out controls at site entrances, sediment basins and traps, enhanced treatments such as polymer or flocculant applications, etc. (see Section 4.4). In application, perimeter barriers are placed along downgradient slopes to intercept sheet flow, while inlet protections such as silt socks or rock filters prevent sediment entry into stormwater infrastructure. Stabilized construction entrances with geotextile underlayment and coarse aggregate reduce sediment tracked onto roadways. At larger sites, sediment basins provide controlled detention, allowing suspended solids to settle before discharge. These measures operate most effectively as an integrated system, reducing off-site sediment transport during peak runoff events.

For example, during commercial site development, silt fence can be installed along the downgradient perimeter to intercept sheet flow and trap suspended sediment. Curb inlet protection using sand-filled bags can be placed around storm drain openings to prevent sediment laden runoff from entering the storm sewer system. A construction entrance with geotextile fabric underlay and coarse aggregate can be used to reduce track-out by heavy equipment and trucks, and a sediment basin can be constructed at the low point of the site to collect and treat runoff from multiple drainage areas.

6.4 Innovative EPSC Measures

During construction, EPSC measures specified in the SWPPP can become less effective, overwhelmed, or useless when challenged by improper installation, inadequate maintenance, or storm events that exceed design capacity. These failures reduce system resiliency, allowing sediment laden runoff to leave the site, degrade downstream water

quality, and create regulatory compliance risks. To address these limitations, recent efforts have focused on integrating innovative approaches into the design and implementation of EPSC measures. Many of these techniques emerged from research targeting common deficiencies observed after storm events, with the goal of enhancing system performance, reliability, and resilience. The following sections describe several innovations that have been scientifically evaluated and demonstrated to improve effectiveness under field conditions.

6.4.1 Silt Fence Innovations

Silt fences remain the most widely used perimeter control measure on construction sites due to their low cost and small footprint. Perhaps the greatest silt fence challenge is the need for an environmentally friendly, plastic-free alternative (Whitman et al., 2025). However, field performance is also a pressing issue that has long been limited by two persistent deficiencies: structural instability and inefficient dewatering. Conventional installations often fail under hydrostatic loading when posts deflect or spacing is excessive, and geotextile pores frequently clog, extending ponding durations well beyond design expectations and time under stress. These shortcomings compromise site operations, increase the risk of bypass flows, and reduce overall system reliability.

To address both issues, Whitman et al. (2021) introduced a plywood dewatering board addition to silt fence installations. The design of the dewatering board provided controlled effluent, reduced the time silt fence was under maximum stress, while preserving sediment retention and water quality standards. Results demonstrated that the silt fence equipped with a dewatering board achieved a 96% sediment retention rate, which was acknowledged to be consistent with previously reported values for conventional silt fences. The silt fence with dewatering board system was able to maintain consistent sediment capture while substantially reducing dewatering times from 24 hours to approximately four hours. For stability, reduced post spacing, reinforced fabrics, and optimized installation geometry substantially increase load-bearing capacity. Structural testing in this work indicated post spacing of four to eight feet for steel posts (depending on size) and five feet for wood posts to reduce sagging, overtopping, and fabric failure (Whitman et al., 2021).


Photo courtesy of J. B. Whitman

Together, these innovations demonstrate importance of integrating practical engineering enhancements into traditional sediment control measures. The proposed dewatering board offers a low-cost, easily implemented solution that significantly improves silt fence performance in both structural stability, dewatering efficiency, and storm resiliency. The dewatering board monitored herein was constructed by drilling four one inch holes along the centerline at heights of three, six, nine, and 12 inches above the ground surface elevation, cutting a 90° angle V-notch weir in the board at an elevation of 18 inches above the ground surface elevation, and attaching it to the silt fence application with metal T-posts, staples, and heavy duty zip ties at the lowest elevation point. More importantly, these improvements can be achieved without major modifications to existing installation practices or extensive additional training for field personnel.

6.4.2 Wattle Ditch Check Dam Innovations

Wattles are commonly employed as ditch check dams and perimeter barriers largely due to low material costs and straightforward installation. However, conventional installation techniques frequently compromise performance, particularly under concentrated flow conditions (i.e., as ditch checks). Standard practice typically involves driving wooden stakes through the wattle, which punctures the netting, accelerates material degradation, and diminishes structural integrity. This method also increases buoyancy, allowing flow beneath the wattle and promoting localized erosion. Although trenching has been suggested to limit undercutting, inspections and visual observations have indicated it can be counterproductive by reducing ground contact and creating preferential subsurface flow paths, further undermining wattle efficacy.

Research has identified several design enhancements that improve wattle performance without adding installation complexity (Donald et al., 2013). A-frame staking (driving stakes at opposing angles on either side of the wattle) anchor wattles securely while preserving netting integrity and wattle compaction. Placing a geotextile underlay or heavy RECP beneath the wattle protects the channel bed from scouring when the system is overtopped. Additionally, securing the wattle with U-shaped sod staples ensures intimate contact with the underlying layer, minimizing flow bypass and undercutting. Finally, curving the ends of the wattle upstream helps to direct water toward the center of the ditch check and reduces the likelihood of flow bypassing around the edges.



Collectively, these improvements extend the service life of wattles, increase sediment capture efficiency, protect channel bottoms from high-velocity flows, and reduce maintenance requirements, enhancing their reliability as an effective EPSC measure.

6.4.3 Inlet Protection Practice Innovations

Silt fence barriers are one of the most common inlet protection practices because of their material availability and straightforward installation. Despite their prevalence, they are not recommended in Tennessee due to inconsistent field performance. Failures are typically associated with inadequate structural integrity, limited impoundment capacity, excessive detention times, or rapid overtopping and bypass. Nevertheless, large-scale testing has shown that with structural enhancements and thoughtful design, silt fence inlet protection can achieve high levels of performance (Perez et al., 2015). In such cases, reinforced configurations are advisable to overcome the deficiencies of standard silt fence inlet protection installations.

To improve structural stability, Perez et al. (2015) tested a system that combined T-posts with 2 × 4 lumber bracing, including perimeter and diagonal members to create a rigid, self-supporting frame. This design resisted structural deformation and maintained impoundment across a 31 foot test channel. However, similar to Section 6.4.1, dewatering is necessary to prevent clogging in silt fence pores and excessively long impoundment times. Perez et al. (2015) recommended a vertical 2 × 4 dewatering board installed with a series of staggered holes ranging from 0.25 to 1.5 inches in diameter, increasing from bottom to top (shown in the photo to the right). This configuration provided slower release of the most turbid, sediment laden water near the base while allowing higher flows to discharge from the upper portion of the impoundment. The staged-release board successfully balanced detention and drainage, preventing prolonged flooding while preserving high sediment capture efficiency of silt fence.


Photo courtesy of J. B. Whitman

The reinforced T-post and lumber installation with the dewatering board was identified as the most feasible and effective installation for silt fence inlet protection. It maintained impoundment across the full channel width, provided effective structural resistance, and fully dewatered within 90 minutes. Compared with conventional silt fence inlet protection practices, this design markedly increased impoundment capacity, reduced risk of catastrophic failure, and balanced sedimentation potential with acceptable drawdown times. These results demonstrate that with modest structural enhancements and the addition of a controlled dewatering mechanism, silt fence inlet protection can function as a reliable and cost-effective EPSC measure.

6.5 Problem Solving

Although EPSC measures are widely implemented, construction sites frequently encounter recurring deficiencies arising from improper installation, insufficient maintenance, site specific limitations, and inadequate planning. Typical failure modes include compromised silt fences, unstabilized soils, sediment laden runoff bypassing control structures, and inadequately protected inlets and conveyance channels. Addressing these challenges requires strategies that are responsive to site conditions and aligned with regulatory criteria.

Effective problem solving involves not only the selection of appropriate control measures but also a clear understanding of how critical variables, such as slope gradient, soil erodibility, rainfall intensity and duration, drainage area size, and construction sequencing, govern erosion and sediment transport dynamics. Proactive planning and timely implementation are essential, supported by continuous monitoring to verify functionality and identify performance deficiencies. Maintenance must be performed promptly to restore effectiveness, particularly following high intensity rainfall events. Equally important is the capacity for adaptive management. As site conditions evolve, EPSC practices must be reevaluated and refined to maintain performance. Field observations, coupled with engineering judgment, provide the basis for modifications that ensure measures remain effective throughout all phases of construction. By systematically diagnosing common EPSC failures and applying targeted, evidence based solutions, project teams can minimize environmental impacts and maintain regulatory compliance.

6.5.1 Inadequate Perimeter Controls

Silt fence barriers are among the most frequently employed perimeter controls on construction sites, yet they are also among the most frequently misapplied. Performance deficiencies typically stem from improper installation, unsuitable placement, and inadequate maintenance. A common failure mode is undercutting, where insufficient trenching permits stormwater to erode beneath the fabric. Similarly, placement within concentrated flow paths often leads to overtopping or structural blowouts during storm events. End sections are

frequently left unsecured or oriented downslope, creating preferential flow paths around the installation. Equally important is recognizing the intended scale of application. Silt fences are designed to treat relatively small drainage areas, typically no more than 0.25 acres of disturbed soil per 100 feet of silt fence. When used to intercept runoff from larger disturbed areas, flow volumes and velocities often exceed barrier capacity, resulting in overtopping, blowouts, and accelerated failure. Compounding these issues, fences often degrade prematurely due to ultraviolet exposure, geotextile clogging, or neglect of inspection and maintenance following rainfall events.

These recurring shortcomings have prompted the development of revised strategies and best practices. To optimize performance, guidance documents consistently emphasize installing silt fence along contour lines, with the fabric trenched to a minimum of six inches and backfilled with compacted soil to ensure a sealed ground interface. J-hooks or end returns should be constructed at specified intervals and at the terminus of each fence run preventing preferential flow paths. Routine inspections are critical to sustain effectiveness, particularly following storm events when fabric tears, displaced posts, or excessive sediment accumulation are most likely. Prompt maintenance to repair or replace damaged sections preserves the functional integrity of the barrier. Without such upkeep, structural degradation and bypass flow rapidly diminish performance.

When properly designed, installed, and maintained, silt fences can provide effective last-line perimeter control, intercepting sheet flow, promoting impoundment, and facilitating onsite sediment deposition.

6.5.2 Unstable Soil and Delayed Seeding

Unstabilized soil and delayed seeding are among the primary drivers of erosion and sedimentation on construction sites, particularly in regions such as Tennessee where frequent rainfall can rapidly mobilize exposed soil. When grading or clearing activities are not promptly followed by stabilization, bare soils are highly susceptible to sheet erosion, rill formation, and sediment laden runoff. These risks are amplified on steep slopes and large disturbed areas that remain idle due to poor scheduling, adverse weather, or inadequate oversight. Prolonged soil exposure not only accelerates onsite erosion but also diminishes the effectiveness of perimeter controls. Uncontrolled runoff can exceed the design capacity of silt fences, sediment basins, and other sediment control measures, leading to bypass and failure. Consequently, regulatory frameworks mandate that disturbed areas be stabilized as soon as practicable.

Best practices emphasize rapid establishment of temporary cover following disturbance. Common approaches include seeding and mulching immediately after grading, phasing

construction to minimize the area of exposure at any given time, and installing ECBs or sod on steep slopes and in drainage channels. Where seeding is not seasonally feasible, hydromulch or bonded fiber matrix products, or temporary vegetation can provide effective interim stabilization. These proactive stabilization measures, when implemented consistently, reduce erosion risks, protect water quality, and help projects remain compliant with TDEC stormwater regulations.

6.5.3 Construction Exits

Construction exits, also known as stabilized construction entrances, are critical EPSC measures designed to minimize the tracking of sediment and mud from construction sites onto public roads. Despite their widespread use, poorly designed or improperly maintained exit pads frequently fail to prevent offsite sedimentation, leading to clogged storm drains, unsafe road conditions, and violations of local and state environmental regulations. Common problems include using insufficient stone thickness or size, failing to install geotextile fabric beneath the stone, and constructing pads that are too short to remove sediment from vehicle tires. Additionally, when construction exit pads are installed on slopes or allow runoff to flow across them, they often become saturated and ineffective, allowing mud to bypass the pad entirely. Without regular maintenance, stone can become compacted or buried under sediment, eliminating its ability to dislodge soil from vehicle tires.

To address these deficiencies, stabilized construction exits must conform to detailed design specifications. Pads should be constructed of clean, uniformly graded crushed stone of sufficient size to promote tire flex and dislodge sediment and placed over a geotextile underlay to prevent subgrade contamination. Dimensions must accommodate the largest vehicles on site, with lengths adequate to allow multiple tire rotations and widths sufficient for safe ingress and egress. Pads should be installed on level ground where feasible, with diversion swales or berms directing runoff away from the entrance and adjacent roadways. Where high traffic volumes or persistent tracking are anticipated, supplemental measures such as tire wash stations are recommended. Routine inspections, sediment removal, and stone replenishment are essential to sustain pad effectiveness and minimize offsite sediment transport.

6.5.4 Channel and Slope Erosion

Channel and slope erosion are major concerns on construction sites as intense rainfall can quickly transform exposed slopes and temporary drainage channels into sources of significant sediment loss. Common problems include failing to stabilize channels before directing water into them, allowing concentrated flows to accelerate down steep or unvegetated slopes, and neglecting to implement temporary flow control measures. These oversights often lead to rill and gully formation and headcut migration, undercutting of

channel banks, and excessive sediment loading downstream. Additionally, construction activities can inadvertently alter natural drainage patterns or increase flow volumes, compounding erosion risks if appropriate controls are not in place. In many cases, sediment basins and silt fences become overwhelmed due to upstream channel or slope failures, reducing their effectiveness and increasing maintenance burdens.

To address these challenges, several EPSC measures can be utilized to reduce flow velocity and protect disturbed soil surfaces. On slopes, strategies include minimizing the area and duration of exposure, using tracking to disrupt runoff, shortening slopes, and immediately applying temporary seed and mulch. RECPs, particularly environmentally- and wildlife-friendly alternatives, are recommended for steeper grades or high flow areas, providing both mechanical protection and a medium for vegetation growth. In drainage channels, temporary check dams made of stone, wattles, or compost logs can be used to slow runoff, trap sediment, and prevent downcutting. Stone must be properly sized for stability. Channel linings, such as TRMs, sod, or (conventional, soil, or void-filled) riprap, can be utilized based on flow intensity and soil type. Energy dissipaters, like level spreaders, rock aprons, or preformed scour holes, can be placed at outfalls to reduce uncontrolled scour at discharge points. These measures, when combined with proper grading, phased construction, and routine inspections and maintenance, form a comprehensive strategy to control channel and slope erosion.

6.5.5 Unprotected Storm Drain Inlets

Unprotected storm drain inlets are a frequent source of sediment discharge from construction sites, often leading to clogged drainage systems, localized flooding, and downstream water quality violations. During grading and earthwork operations, runoff from disturbed areas can carry large volumes of sediment directly into nearby curb inlets, drop inlets, or area inlets if effective inlet protection measures are not installed. Common problems include using inadequate materials or failing to secure devices properly, allowing runoff to bypass the protection altogether. Inlet controls are also frequently undersized, not maintained after storm events, or removed prematurely before final site stabilization. These failures reduce the effectiveness of the entire erosion and sediment control system and place projects at risk of non-compliance.

To combat these issues, guidelines recommend several well-tested strategies for inlet protection based on inlet type and flow conditions. For curb inlets, gravel-filled bags or block and gravel barriers are often used to filter runoff while allowing overflow through the top. Drop inlets are commonly protected with gravel and hardware cloth baskets or sediment filter logs arranged in a ring around the opening (Section 4.4.4). All devices must be installed with adequate ponding area upstream and must not create traffic hazards or block

emergency drainage. Maintenance is critical, protection measures must be inspected after rain events, with sediment and debris removed before the next storm. In high flow or paved areas, reinforced inlet devices or manufactured solutions may be needed.

6.5.6 Sediment Basins on Rocky Sites

Constructing sediment basins on rocky or shallow bedrock sites presents unique challenges that can compromise performance if not properly addressed. One of the most common problems is the inability to excavate the basin to the required depth due to hard subsurface conditions, which limits storage volume and reduces basin capacity. Additionally, achieving a proper seal at the bottom of the basin can be difficult, leading to underdrainage and turbidity as water seeps through rock fractures. Embankment construction can also be problematic when suitable soil is not available on site, making it difficult to compact berms or form stable spillways. Furthermore, installing outlet structures, such as risers and skimmer systems, into uneven or unstable rock surfaces may lead to leaks, misalignments, or structural failure under high flow conditions. These issues are often compounded by poor site planning or rushed installation schedules that do not allow time for necessary excavation or adaptation.

To address these challenges, guidelines recommend several adaptive strategies for sediment basin construction on rocky terrain. If full excavation is not feasible, designers can modify basin geometry to include wider, shallower configurations that maintain storage volume through increased surface area or the adoption of multiple measures used in a treatment train to achieve equivalency (Section 5). Where seepage is a concern, geosynthetic liners or clay blankets can be used to seal the bottom of the basin. For embankments, contractors can import suitable fill material or use rock-core berms wrapped in geotextile fabric to maintain structural integrity. Outlet structures can be stabilized with concrete pads or anchor systems to prevent shifting, and emergency spillways can be armored to handle overflows. Regular inspections, maintenance, and as-built verifications are essential to ensure the basin is functioning as intended. These strategies allow for effective sediment control even on difficult sites.

6.6 Compliance and Enforcement

The SWPPP, EPSC plans, and inspection reports serve as CGP compliance records for the permittee and must be retained at the construction site or another location accessible to TDEC-DWR. Instances of noncompliance may arise from out-of-date documents, lack of maintaining measures, inappropriately placed measures, or ineffective and incorrectly installed measures. Such instances may be considered a permit violation. When a noncompliance event occurs, TDEC-DWR may issue a notice of violation (NOV) letter to the permittee requesting prompt corrective actions be taken, remediating any sediment loss.

The primary objective of TDEC is to achieve permit compliance and prevent pollutant discharge to waters of the state; enforcement actions can generally be avoided through timely cooperation and corrective actions. In general, three response scenarios apply when sediment is discharged from a construction site. First, if sediment is released offsite to a downgradient property (i.e., a non-waterbody), the permittee and/or contractor must remove the discharged material, evaluate the cause of the release, and repair, maintain, or install additional EPSC measures to prevent recurrence. Notification to TDEC is not required in this scenario; however, the discharge, why the discharge occurred, and all corrective actions must be documented in the site inspection records. Second, if a sediment discharge reaches a receiving water and is of sufficient magnitude to require remediation or restoration, both submission of an ARAP and notification to TDEC are required. Notification to the TDEC-DWR must describe the severity of the discharge, indicate that an ARAP will be (or was) submitted for waterway maintenance, identify the cause of the release, outline proposed measures to prevent future occurrences, and provide a timeline for EPSC measure maintenance or improvements. Finally, if a discharge results in the delivery of sediment, increased turbidity, or an objectionable color contrast to a receiving water but does not warrant remediation or restoration, notification to TDEC is not required. In this case, the inspector, permittee, and/or contractor must investigate the cause of the discharge, implement or improve EPSC measures to prevent recurrence, and document the release and all corrective actions in the site inspection logs. All inspection records must be made available upon request or during site visits.

There are some instances where compliance with the CGP is maintained, but sediment export was observed. This often results from events exceeding the design storm. If it can be documented that onsite rainfall depth exceeded the design storm (recorded from NOAA Atlas 14), and the SWPPP, EPSC plans, and inspection reports have been maintained, the sediment release may not be a compliance issue. The design storm intensity can also be exceeded and verified with NOAA Atlas 14. In this case, the duration used is equal to the T_C (not 24 hours) of the watershed draining to the outfall in question. Under these scenarios where a site compliance is maintained during a sediment release, the permittee is still responsible for remediating any sediment loss.

In certain cases, TDEC may require submission of a corrective action plan (CAP) detailing measures to be taken to address specific violations. Prior to formal enforcement, TDEC-DWR may schedule a compliance review meeting with the permittee to establish a coordinated plan for achieving compliance. These meetings are generally held following recurring NOVs to the permittee for failing to achieve compliance. Continued or repeated noncompliance may result in formal enforcement proceedings. In such cases, TDEC-DWR may conduct a Natural Resource Damage Assessment (NRDA) to determine appropriate monetary penalties

to offset impacts to state waters. In addition, TDEC-DWR may require the permittee to attend EPSC related educational/training courses.

Persistent issues with inadequate design may stem from substandard engineering practices. For example, cost-saving design decisions may compromise environmental protection and compliance. Although engineers are not typically subject to financial penalties under the CGP, they are licensed and regulated by the Tennessee Board of Architectural and Engineer Examiners. Licensure status may be verified, and formal complaints submitted, through the Tennessee Department of Commerce and Insurance online portal at https://access.cloud.commerce.tn.gov/portal/public.

TDEC-DWR is the NPDES permitting authority responsible for regulating water quality under the CWA. However, many stormwater related complaints received by TDEC pertain to flooding, which falls outside of such regulatory authority. In such cases, disputes are typically resolved under civil drainage law, which states a landowner may not:

1. Obstruct the natural flow of water;
2. Increase the natural volume of runoff;
3. Accelerate the natural velocity of flow; or
4. Change the direction of the natural flow of runoff.

Details of natural drainage laws can be found at https://mrln.mtas.tennessee.edu/se/final/Portal/Default.aspx?component=b-USER&record=d36502cd-ecc1-44e4-a67a-d6648dac2502.

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Chapter 7 - Conversion of Temporary Controls to Permanent Stormwater Treatment

7.1 Background

This section provides guidance on converting temporary sediment controls into permanent stormwater control measures (SCMs). The recommendations are intended as a starting point and should be applied with sound engineering judgment. Final SCM designs must follow recognized best practices and be developed in accordance with applicable local and state stormwater ordinances and their design manuals.

Construction EPSC measures and permanent SCMs serve complementary but distinct purposes within development projects. Certain EPSC practices possess design characteristics that can be readily adapted for long-term stormwater management. For example, a sediment basin constructed for temporary sediment storage and sized to accommodate both construction and post construction requirements, can later be retrofitted with a permanent outlet structure to function as a stormwater wet or dry pond. This dual use allows for more efficient use of land and resources. Among the most suitable EPSC measures for conversion are sediment basins, swales, and other conveyances. Guidance on converting these practices is provided in the following sections.

7.2 General Considerations

Successful conversion of temporary sediment controls into permanent SCMs requires careful attention to planning, construction timing, and sediment removal. Each of these elements plays a critical role in ensuring that the permanent system functions as intended, avoids premature failure, and complies with design and regulatory requirements. As mentioned in Section 4.4.7, the sediment storage zone of a sediment basin can be over-excavated, such that accumulated sediment during the construction phase can bring the bottom elevation to its permanent grade. This method reduces sediment removal maintenance and is encouraged if the accumulated sediment is stabilized before the basin is converted to a permanent measure.

Planning

Pre-construction planning is essential for an effective conversion. Temporary EPSC measures should be aligned to convey runoff in the same direction as the intended permanent discharge point, ensuring a smooth transition post-construction. Design calculations must account for both construction-phase and long-term regulatory flow requirements, with EPSC measures sized or adaptable to meet those needs. Construction plans should clearly

document conversion details, including access provisions for equipment and modifications to outlet structures. In many cases, outlet structures must be designed for flexibility, such as the ability to dewater or adjust orifice locations using knock-out ports, so they can be adapted during conversion without extensive reconstruction. Further, modifications to site conditions must also be documented. For example, temporary construction EPSC measures may have lesser designs if diversions have been installed upstream to limit the associated drainage area. If those diversions are also temporary and are to be removed upon site stabilization, the permanent SCM must account for the increase in drainage area.

Construction Timing

Conversion must only take place after site construction is complete, and disturbed areas have achieved final stabilization requirements. This precaution minimizes the risk of sediment laden runoff entering the permanent SCM, where it could clog treatment components or reduce storage capacity. If an earlier conversion is necessary, equivalent sediment controls may be used to stay in compliance with the CGP. To the extent possible, conversion is to be scheduled during dry weather forecasts, reducing the likelihood that runoff will enter the system before it is fully operational. Rapid execution of conversion activities is critical to protect water quality and maintain design functionality (which includes quickly establishing vegetative cover).

Sediment Removal

Prior to conversion, all accumulated sediment in the EPSC measure must be removed to achieve the intended design elevations of the permanent SCM. Dewatering strategies should be incorporated into the plan to allow safe and efficient removal of ponded water and sediment. Sediment deposits are often fine-grained and may not be suitable for plant establishment making disposal or stabilization necessary. Providing a clean, stable foundation ensures that vegetation, outlet structures, and other design elements of the permanent control can perform as intended over the long term.

7.3 Sediment Basin Conversion – Retention Basin

Adapted from Tschantz (2012), the following steps outline the conversion process of a temporary sediment basin into a permanent stormwater detention facility:

1. *Dewatering*: Once the construction site is fully stabilized, the sediment basin is to be dewatered to the top of the sediment layer in both the forebay and the main basin area. Dewatering is to begin after the pond has drained to or below the sediment storage elevation and must occur over the course of at least one day to avoid resuspension of captured sediment, embankment slumping, or slope failure. A

surface skimmer discharging into the riser can be used to lower the live storage pool to the sediment layer. Forebay areas, which trap mostly sands and coarse silts, if properly constructed above the elevation of the sediment storage zone, should dewater by gravity. However, dewatering the mucky bottom of the sediment storage area, containing a greater proportion of finer sediments, may be challenging. Dewatering the slurry in the bottom of the sediment storage zone may require digging out a small pit at the lower end of the basin near the riser and pumping into a silt bag, or pumping/siphoning into a small clarification chamber. Dewatering may require three or four days of dry weather and a few additional days for the drained sediment to sufficiently dry out for handling.


Source: TNWRRC

2. *Demucking*: Following dewatering, sediment that has accumulated in the forebay and main basin during the construction period must be removed and disposed of without allowing sediment to enter into the outlet structure, local storm sewer drainage system, or receiving waters. Once sufficiently dried, sediment may be excavated with a track hoe or loader to the planned retention pond grade and disposed of as fill in accordance with the approved SWPPP. In some cases, wet bottom material may require polymer thickening for more timely and orderly sediment removal. In such cases, distribute the soil-specific anionic polymer uniformly across the sediment surface. Use the bucket of the excavation equipment to mix it into the soil to a depth of up to three feet. Allow ten to 20 minutes for the polymer to interact with the sediment; increased mixing will shorten the reaction time. The polymer binds the sediment, causing it to consolidate and making removal more efficient. To prevent damage, heavy equipment should not be used around the riser, outlet barrel, and foundation.
3. *Sediment Disposal*: Depending on the characteristics of the accumulated sediment, it may be used as fill material. Otherwise, or if polymers were used during the operational lifespan of the sediment basin, a disposal plan ensuring proper disposal of the accumulated sediment must be outlined in the SWPPP and followed.
4. *Removing Sediment Basin Controls*: The forebay of a sediment basin may be kept in place if specified by permanent, post-development design. However, the primary dewatering device assembly for the main sediment basin should be removed, along with the resting pad if a surface skimmer was used. If the skimmer is appropriately sized for the post-construction design, it may be repurposed for use in an extended

retention basin or a similar facility. The riser foundation, trash rack (including any anti-vortex device), and outlet barrel connection must be carefully inspected for structural integrity and prepared to function as a service spillway and peak flow control. Extreme caution is required if the outlet riser structure is to be modified for permanent stormwater control, as improper handling can compromise the integrity of the riser and barrel assembly. In particular, heavy equipment use or coring within the riser structure can lead to cracking or other structural damage. For this reason, any heavy construction work associated with the riser should be completed during its initial installation. Similarly, installing a temporary riser structure for the sediment basin with the intention of later replacing it with a permanent riser after stabilization is to be avoided.

5. *Checking dimensions and inspections*: Surveying the basin verifies all dimensions and elevations to confirm that the permanent retention pond provides the intended storage volume, hydraulic capacity, and freeboard. Earthen embankments require evaluation for settlement, as loss of elevation can compromise the emergency spillway design discharge. The dam crest profile, slopes, and foundation demand close inspection for sagging, depressions, cracks, bulges, or slumping. Particular attention is warranted at the outlet pipe of the spillway, where internal erosion, seepage, or piping may result from loose joints, crushed pipe, displacement of the riser, poor compaction around the pipe barrel, or differential settlement. These conditions threaten long-term stability and must be addressed promptly to prevent failure. Permanent inlet headwalls and scour-prevention measures, including rock riprap and associated geotechnical linings, also require inspection for damage or displacement. Any evidence of instability, structural weakness, inadequate storage capacity, or hydraulic malfunction necessitates repair or reconstruction to restore full design performance.
6. *Installing permanent stormwater controls*: Permanent orifices, weirs, valves, and multi-stage outlet structures must be carefully installed to avoid damaging the spillway. Cover plates and knock-outs are to be removed in accordance with the approved design, and all openings should be verified for correct size and elevation.
7. *Final Stabilization*: The basin bottom, side slopes, embankment crest, and upstream and downstream faces must be permanently stabilized with vegetation or


Source: TNWRRC

other approved measures to prevent erosion and resuspension of sediments. Stabilization measures must be inspected following installation to ensure adequacy.

8. *Close-out*: The close-out process is only necessary within a MS4, though this process is encouraged for all SCMs. The permanent SCM must be inspected and verified to confirm compliance with approved design plans and specifications. The 90-day verification period begins upon completion of the permanent SCM not the sediment basin construction or the beginning of conversion. Permanent maintenance agreements must be established to ensure long-term inspection, sediment removal, and repair. Easements should guarantee access for maintenance by the responsible party. Verify the closeout process of the SCM with the local MS4.

7.4 Sediment Basin Conversion – Detention Basin

The conversion of a sediment basin to a detention basin follows a process similar to that of a retention basin, with the primary distinction being the placement of the permanent discharge orifice. In a detention basin, the orifice is to be at the bottom elevation of the basin to enable complete dewatering between storm events. Establishment of a stable, vegetated basin floor is a critical component of this conversion, as vegetation enhances both water quality treatment and long-term soil stability. The suitability of residual sediments for supporting vegetation requires evaluation; material that is excessively fine-grained, compacted, or lacking in organic matter and nutrients may inhibit plant establishment. Where sediment quality is inadequate, the basin bottom can be excavated to a depth of approximately six inches and reconstructed with suitable topsoil to restore the design elevation. Incorporating topsoil provides improved soil structure, organic content, and nutrient availability, thereby promoting vegetation growth and ensuring proper basin function.

7.5 Sediment Basin / Trap Conversion – Bioretention / Infiltration Basin

Bioretention and other infiltration-based practices rely heavily on the permeability of underlying soils for proper function. Sediments accumulated during construction are often dominated by fine particles that not only exhibit poor drainage characteristics but also have the potential to clog more permeable native soils. If the area does not have a HSG class rating of A or B, methods to enhance infiltration and counteract compaction are highly encouraged. Otherwise, conversion of EPSC measures to permanent infiltration based measures are not recommended. Where such a conversion is pursued, the following procedures provide a framework for preserving infiltration capacity and ensuring long-term functionality:

1. Restrict heavy equipment access to the area designated for infiltration following construction. Maintaining the natural infiltration capacity of in situ soils is critical, and

the designated footprint can be protected through the use of high visibility fencing or equivalent demarcation;
2. Stabilize the contributing watershed prior to initiating basin conversion. Preventing uncontrolled sediment-laden flows is essential to avoid clogging the infiltration media. Temporary diversion of runoff around the infiltration area during construction is recommended;
3. Close-out the sediment basin in accordance with the procedures previously described;
4. Construct the bottom of the sediment basin approximately one foot higher than the final design elevation of the bioretention or infiltration basin. This allows for an additional one-foot excavation to expose uncompacted in situ soils (OEPA, 2025);
5. Perform subsoil decompaction within all infiltration areas. Loosening with a backhoe ripper or equivalent equipment to a depth of at least 18 inches below subgrade reduces construction-related compaction and enhances infiltration. This soil ripping process has demonstrated measurable improvements in infiltration performance; and
6. Verify soil infiltration rates following decompaction and excavation to confirm design assumptions. Where measured infiltration rates are inadequate, the installation of an underdrain may be required, particularly for bioretention systems.

7.6 Swale Conversion – Grassy Swale

The conversion of temporary swales used as an EPSC measure to permanent grassy swales is generally a straightforward process when properly planned during design and construction. Aligning the swale in the same direction and slope as the final configuration minimizes the need for regrading and reduces land disturbance during transition. The final swale slope should not result in erosion which is dependent on the vegetation cover type (Tables 3.3.2.1-A,B). Design procedures should follow those in Sections 4.2.6.1 and 4.3.2. The following considerations support a successful conversion:

1. Remove all accumulated sediment from the swale to restore conveyance capacity and prepare the channel for permanent stabilization;
2. Re-establish or modify channel geometry and longitudinal slope as needed to achieve the design flow capacity;
3. Evaluate soil quality in the swale bottom prior to establishing permanent vegetation. Adequate organic matter and nutrient levels are necessary to support vegetation growth. Where soils are deficient, the bottom may be excavated four to six inches and backfilled with topsoil to the intended design grade.

Where conversion to a bioswale (a linear bioretention system) is desired, additional design considerations apply. Bioswales incorporate engineered media beneath the channel bed to

promote infiltration as stormwater flows downslope. In such cases, the swale bottom is excavated an additional one to two feet and replaced with bioretention media. Subsoil decompaction, as previously described, is critical to restore infiltration capacity. The surface of the bioswale may be stabilized with sod or decorative stone to limit erosion and provide rapid establishment of the system.

References:

OEPA. (2025). *Rainwater and Land Development: Post-Construction Stormwater Management Structural Practices.*

Tschantz, B. A. (2012). *Converting temporary sediment basins to permanent stormwater detention ponds in Tennessee.* ASCE-TSPE-ACEC Joint annual meeting.

Appendices

Appendix A Example Drawings

Appendix B SWPPP Templates for Small Sites

Appendix C Checklists and Forms

Appendix D Seed Mix Examples

Appendix E Definitions

Appendix D - Seed Mix Examples

The Seed Mix Appendix is intended to provide engineers and contractors with generalized examples of seed mixes suitable for a variety of projects. The tables contained in this Appendix offer sample mixes for stabilizing diverse sites; however, they are not tailored to specific site conditions. It is strongly encouraged that interested parties consult with qualified professionals and/or provided references to develop site-specific seed mixes that address the unique characteristics and requirements of each location.

Disclaimer:
The sample seed mixes included in this appendix are generalized for use across the state and are not guaranteed to achieve site stabilization. Furthermore, their use may not ensure compliance with additional or local regulatory requirements.

Regions of the State

Tennessee is divided into regions that reflect the diverse ecology, soils, and climate of the state. For the purposes of this guidance document, the regional divisions align with those established by TDOT (Figure D-1). This alignment is intended to support consistency across the state and enhance ease of use for contractors and other stakeholders.

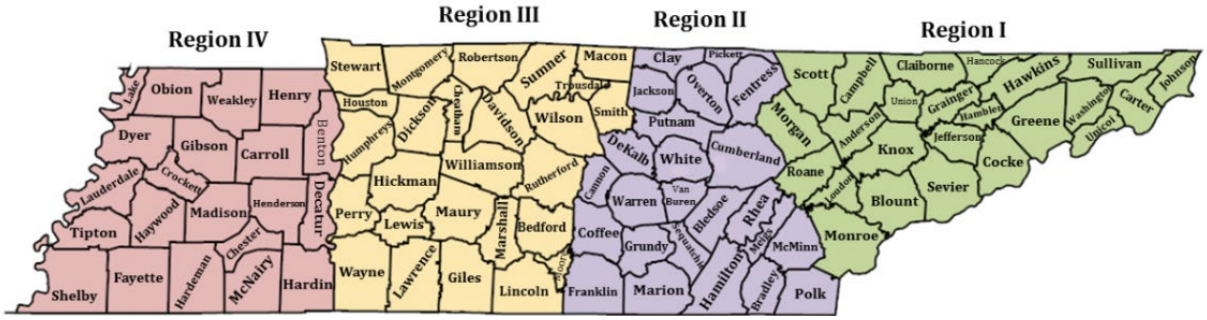


Figure D-1: Map of TDOT regions. Source: TCSA.

In addition to TDOT regions, Tennessee contains various and distinct ecoregions (Figure D-2), which play a significant role in determining the suitability and success of plant species used for site stabilization. While ecoregions are not the primary focus of the guidance herein, their influence on vegetation success warrants acknowledgment. Arnwine and Brame (2007), bplant, and the EPA provide additional details on Tennessee ecoregions.

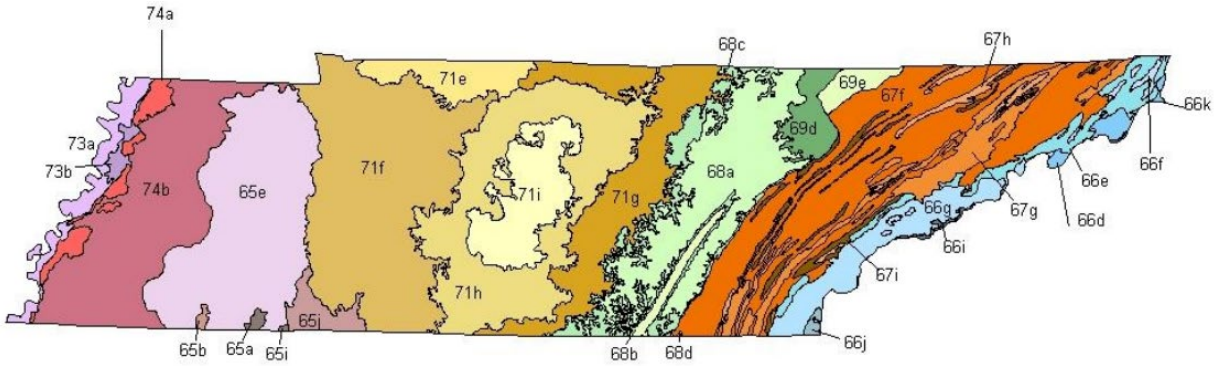


Figure D-2: Map of Tennessee Level IV ecoregions. Source: Arnwine and Brame (2007).

For simplicity, it is assumed that TDOT Region I contains ecoregions 66 (Blue Ridge), 67 (Ridge and Valley), 68 (Southwestern Appalachians) and 69 (Central Appalachians); TDOT Region II contains ecoregions 66 (Blue Ridge), 67 (Ridge and Valley), 68 (Southwestern Appalachians) and 71 (Interior Plateau); TDOT Region III contains ecoregions 65 (Southern Plains) and 71 (Interior Plateau); and TDOT Region IV contains ecoregions 65 (Southern Plains), 71 (Interior Plateau), 73 (Mississippi Alluvial Plain), and 74 (Mississippi Valley Loess Plains 3). However, the Tennessee ecoregion interactive map (UTK) is suggested for use to more easily and precisely identify the specific ecoregion in which a specific project is located.

Vegetation as Erosion Prevention

Within Chapter 4 Section 4.2.6 *Stabilization* of this handbook background is provided on erosion prevention through stabilization. When soil is stabilized with vegetation, the stormwater is unable to dislodge the sediment particles and carry them downstream and off-site. Vegetative stabilization is a long-term fix to soil loss. The root system of vegetation is paramount in the prevention of erosion. Grass roots tend to fall into the following categories: fibrous, rhizomes, and stolons.

Fibrous roots spread outward and downward from the plant's base. The roots provide a dense web of fine tendrils that allow the plant to efficiently absorb water and nutrients from the soil.

Rhizomes create a dense turf just underneath the soil using stems and nodes. They grow horizontally and spread rapidly, creating interconnected coverage.

Stolons are similar to rhizomes and use the same method of stems and nodes to grow; the difference is that stolons spread horizontally above the soil. They are resistant to weeds and recover from damage quickly due to their dense matting. A very erosion resistant surface is provided once established.

Planting Seasons Across the State

Successful planting of all vegetation is critically dependent on planting times and season. Fortunately, temporal variations of seasons across the state are marginal. Thus, regardless of TDOT Region or ecoregion, optimal planting times for introduced seed species are relatively consistent across the state. Warm season species are best planted from May to August while cool season species are best planted in November to February (Table D-1). During the transition of seasons (March, April, September, and October), the optimal species of plant may vary.

Table D-1: Optimal planting months for cool season introduced plant species (C), warm season introduced plant species (W), and plant species that may prosper during months of transitioning seasons (T).

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
C	C	T	T	W	W	W	W	T	T	C	C

The diverse climate of Tennessee allows for the successful growth of both cool-season and warm-season turfgrasses, with selection depending on factors such as geographic location, site use, and irrigation access. Warm season grasses typically thrive in west Tennessee and in areas with full sun. Further they can withstand heavy foot traffic and most actively grow from late spring through fall (optimal temperatures: 80–95°F). In contrast, cool-season grasses are better suited for sites in east Tennessee, higher elevations, or areas with partial shade, as they peak in spring and fall when temperatures are more moderate (optimal temperatures: 60–75°F). Warm season and cool season turfgrass species are provided in Table D-2. A-LIST, NTEP, TWCA, and UTIA are additional resources for turfgrass traits and resilience, turfgrass certifications, turfgrass conservation goals, and science based turfgrass information, respectively. Review of these resources may be necessary when turfgrass seed mixes have not been implemented or successful in particular areas.



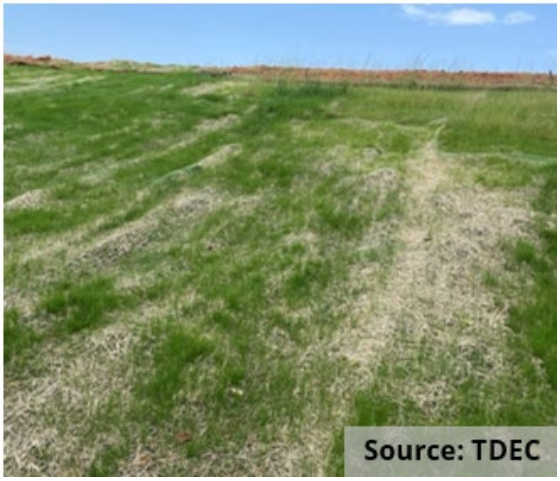


Table D-2: Warm and cool season turfgrasses and temporary stabilization species.

Warm Season	Cool Season
Turfgrasses	Turfgrasses
Bermudagrass (*Cynodon spp.*)	Tall Fescue (*Festuca arundinaces*)
Zoysiagrass (*Zoysia spp.*)	Fine Fescues (*Festuca spp.*)
Centipedegrass[a] (*Eremochloa ophiuroides*)	Perennial and Annual Rye Grass (*Lolium spp.*)
Carpetgrass[a] (*Axonopus compressus*)	Kentucky and Hybrid Bluegrass (*Poa spp.)*
St. Augustinegrass[a] (*Stenotaphrum secundatum*)	
Cover Crops	Cover Crops
Sun Hemp (*Crotalaria juncea*)	AW Peas (*Pisum sativum*)
Buckwheat (*Fagopyrum esculentum*)	Wheat (*Genus Triticum*)
Brown Top Millet (*Urochloa ramosa*)	Rye (*Secale cereale*)
Partidge Pea (*Chamaecrista fasciculata*)	Oats (*Avena sativa*)
Sorghum Sudan (*Sorghum x drummondii*)	Crimson Clover (*Trifolium incarnatum*)
TEFF (*Eragrostis tef*)	Annual Ryegrass (*Lolium multiflorum*)
	Hairy Vetch (*Vicia villosa*)

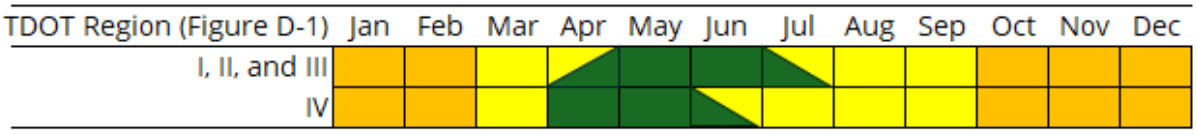
[a]Less commonly used species in Tennessee

A variety of warm and cool season cover crops can be used to provide short-term erosion control and soil protection, depending on the time of year and site conditions (Table D-2). Warm season species such as buckwheat, sun hemp, and brown top millet are typically planted in late spring and summer, while cool season options like oats, rye, and vetch are more appropriate for fall through early spring.

When selecting cover crops, consider pairing a cereal such as wheat or rye with a legume like clover, peas, or vetch. This combination promotes rapid cover and improves soil structure and fertility while providing erosion protection. For more detailed descriptions of these functional groups, consult local extension resources or crop-specific publications. One such resource is *Cover Crop Species Selection for Tennessee Corn and Soybean Systems* by Sykes (2019).

In situations where native vegetation is ultimately desired, temporary cover crops play an important role in facilitating successful native establishment. Native seed mixes should be installed during optimal planting windows specific to TDOT regions, as detailed in Figure D-3. In TDOT regions I, II, and III the ideal planting period for native species is generally from May to July. During this time, standard application methods and seeding rates can be used. Outside of this window, adjustments are required depending on conditions. For example, planting in transitional months require a 25% increase in seeding rate, while non-optimal months require hydroseeding with a flexible growth medium and a 50% increase in rate to compensate for limited growth. As mentioned, it is always best to consult local experts for optimal mixes.





Warm season: drill or broadcast only normal seeding rate		Brown top millet (*Urochloa ramosa*)[a] nurse crop at 4 pounds per acre
Transition: drill, broadcast or hydroseed seeding increased by 25%		Brown top millet (*Urochloa ramosa*)[a] nurse crop at 4 pounds per acre and oats (*Avena sativa*) at 20 pounds per acre.
Non-optimum: hydroseed only with flexible growth medium up to six months of hold seeding rate increased by 50%		Oats (*Avena sativa*)[a] at 32 pounds per acre.

[a]All Native Planting must include a nurse crop.

Figure D-3: Optimal timeframes for planting native species.

Nurse crops are always required when planting native seed mixes. These species provide rapid germination, immediate soil cover, and help suppress weeds while native species establish. Brown top millet is typically used during warm months at a rate of four pounds per acre. In cooler months, oats may be planted at 32 pounds per acre. To ensure adequate coverage during transitional periods, a combination of brown top millet at four pounds per acre and oats at 20 pounds per acre may be warranted. Optimal nurse crop rates and timing are also shown in Figure D-3.

If native planting must occur outside the ideal window, careful planning is needed to ensure stabilization. While growth may be limited in the first season, the nurse crop will provide temporary erosion control and allow time for weed management prior to native emergence. Consider consulting with a native vegetation expert to assess the suitability of dormant seeding or to refine your approach for challenging seasonal conditions. To assist in species selection and seeding rates tailored to your specific site, consider using tools such as the Cover Crop Selector (PSA), which complements the species guidance found in Table D-2 by providing detailed cover crop information based on user defined goals and planting timeline constraints.

Example Seed Mixes

The following tables present general seed mixes that are broadly applicable across the state, generalized to TDOT regions (Figure D-1), and tailored to generalized site conditions (Tables D-4 through D-21). All provided seed mixes minimize the use of invasive species and are non-threatening to native vegetation. If a particular species within a mix is unavailable, substitutions may be made; however, approval from the designer may be necessary for any alternative species or planting rates. When species substitution is necessary, proportionally increase the seeding rate of the remaining grass species to account for the omitted species. The provided seed mixes for streambanks, upland buffers, and wetlands are examples; site-specific mixes are recommended and should be developed in consultation with a seed supplier. Species availability may vary by season and vendor, and wholesale suppliers can tailor custom blends based on site conditions and seed availability, improving the likelihood of successful stabilization. Most seed mixes listed with natives include the total application rate in pure live seed (PLS) which is the computed by multiplying the percent germination by percent purity. Refer to Figure D-3 for ideal planting schedules. Because the provided seed mixes are generalized, additional resources may help to create a site-specific seed mix or achieve additional goals for the site. Such resources are provided in Table D-3.

Table D-3: Seed mix additional resources.

Source	Title
NRCS (a)	*Tennessee Natural Resources Conservation Service*
NRCS (2019b)	*Conservation Practice Standard Critical Area Planting for Tennessee (Code 342)*
NRCS (c)	*Field Office Technical Guide*
TDOT	*Landscape Design Guidelines*
TNIPC	*Landscaping with Native Plants*
TNPS	*Tennessee Native Plant Society Resources Links*

Table D-4: Introduced warm season seed mix for temporary cover broadly applicable across the state.

Common Name	Botanical Name	% by Weight
Annual rye grass	*Lolium multiflorum*	73
Brown top millet	*Panicum ramosum*	27
Total		100

Table D-5: Introduced cool season seed mix for temporary cover broadly applicable across the state.

Common Name	Botanical Name	% by Weight
Annual rye grass	*Lolium multiflorum*	84
Spring oats	*Avena sativa*	16
Total		100

For permanent vegetation of introduced species see Table D-2. This provides a list of monoculture turf grasses. Alternatively, these turf grasses can be mixed with legumes or clovers to provide a more diverse and stable environment (Table D-6 and D-7).

Table D-6: Introduced warm season seed mix for permanent cover broadly applicable across the state.

Common Name	Botanical Name	% by Weight
Bermudagrass	*Cynodon spp.*	60
Brown top millet	*Panicum ramosum*	30
Partidge Pea	*Chamaecrista fasciculata*	10
	Total	100

Table D-7: Introduced cool season seed mix for permanent cover broadly applicable across the state.

Common Name	Botanical Name	% by Weight
Tall fescue	*Festuca arundinacea*	70
Annual rye grass	*Lolium multiflorum*	20
White clover	*Trifolium repens*	10
	Total	100

To promote ecological diversity and immediate stabilization, the integration of native and introduced species can be advantageous. Native species contribute to biodiversity, enhance wildlife habitats, and improve visual appeal. A diverse plant community also increases resilience to disease, reducing the likelihood that a single pathogen could decimate the vegetation. The regional tables present a hybrid of native and non-native species to support both diversity and rapid stabilization. Example seed mixes are provided for dry, wet, and sloped conditions within each region (Tables D-8 through D-16). Each species contributes specific functional traits and root structures, and the inclusion of a variety of species fosters a robust ecosystem. This approach offers benefits that extend beyond water quality improvement and erosion control.



Table D-8: Hybrid seed mix example for dry areas in TDOT regions I and II.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Kentucky 31 fescue	*Festuca arundinacea Schreb*	20	26
Virginia wildrye	*Elymus virginicus*	10	13
Eastern gamagrass	*Tripsacum dactyloides*	10	13
English rye or perennial ryegrass	*Lolium perenne*	15	19.5
White clover	*Trifolium repens*	2	2.6
Red clover	*Trifolium pratense*	2	2.6
Little bluestem	*Schizachyrium scoparium*	8	10.4
Side-oats grama	*Bouteloua curtipendula*	8	10.4
Purple lovegrass	*Eragrostis spectabilis*	2	2.6
Grasses subtotal		77	100
Annual ryegrass	*Lolium multiflorum*	30	
Cover crop subtotal		30	
Total PLS		107	100

Table D-9: Native seed mix example for wet areas in TDOT regions I and II.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Common wood sedge	*Carex blanda*	0.94	3.8
Yellowfruit sedge	*Carex annectens*	0.94	3.8
Fox sedge	*Carex vulpinoidea*	0.94	3.8
Common rush	*Juncus effuses*	1.25	5
Switchgrass	*Panicum virgatum*	0.94	3.8
Grasses, sedges, and rushes subtotal		5	20
Swamp milkweed	*Asclepias incarnata*	0.94	3.8
Common boneset	*Eupatorium perfoliatum*	0.75	3
Common sneezeweed	*Helenium autumnale*	0.88	3.5
Virgnia mountain mint	*Pycanthemum virgnianum*	0.88	3.5
Swamp verbena	*Verbena hastata*	0.75	3
Golden zizia	*Zizia aurea*	0.81	3.3
Forbs subtotal		5	20
Annual ryegrass	*Lolium Multiflorum*	15	60
Cover crop subtotal		15	60
Total PLS		25	100

Table D-10: Native seed mix example for sloped areas in TDOT regions I and II.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Side-oats grama	*Bouteloua curtipendula*	1	4
Common wood sedge	*Carex blanda*	0.88	3.5
Noddling wild rye	*Elymus canadensis*	0.88	3.5
Purple lovegrass	*Eragrostis spectabillis*	1	4
Little bluestem	*Schizachyrium scoparium*	1.25	5
Grasses, sedges, and rushes subtotal		5	20
Buttefly milkweed	*Asclepias tuberosa*	0.94	3.8
Lencelead coreopsis	*Coreopsis lanceolate*	0.75	3
Wild bergamot	*Monarda fistulosa*	0.81	3.3
Beard tongue	*Penstemon digitalis*	0.81	3.3
Gray-headed coneflower	*Ratibida pinnata*	0.81	3.3
Smooth blue aster	*Symphyotrichum leave*	0.88	3.5
Forbs subtotal		5	20
Annual ryegrass	*Lolium multiflorum*	30	60
Cover crop subtotal		30	60
Total PLS		35	100

Table D-11: Hybrid seed mix example for dry areas in TDOT region III.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Kentucky 31 fescue	*Festuca arundinacea Schreb.*	20	26
Virginia wildrye	*Elymus virginicus*	10	13
Easter gamagrass	*Tripsacum dactyloides*	10	13
English rye or perennial ryegrass	*Lolium perenne*	15	19.5
White clover	*Trifolium repens*	2	2.6
Red clover	*Trifolium pratense*	2	2.6
Little bluestem	*Schizachyrium scoparium*	8	10.4
Side-oats grama	*Bouteloua curtipendula*	8	10.4
Purple lovegrass	*Eragrostis spectabilis*	2	2.6
Grasses subtotal		77	100
Annual ryegrass	*Lolium multiflorum*	30	
Cover crop subtotal		30	
Total PLS		107	100

Table D-12: Native seed mix example for wet areas in TDOT region III.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Common wood sedge	*Carex blanda*	1	4
Yellowfruit sedge	*Carex annectens*	1	4
Fox sedge	*Carex vulpinoidea*	1	4
Common rush	*Juncus effuses*	1.25	5
Switchgrass	*Panicum virgatum*	0.94	4
Grasses, sedges, and rushes subtotal		5	20
Swamp milkweed	*ASclepias incarnata*	0.94	4
Common boneset	*Eupatorium perfoliatum*	0.75	3
Common sneezeweed	*Helenium autumnale*	0.88	4
Virginia mountain mint	*Pycanthemum virgnianum*	0.88	4
Swamp verbena	*Verbena hastata*	0.75	3
Golden zizia	*Zizia aurea*	0.81	3
Forbs subtotal		5	20
Annual ryegrass	*Lolium multiflorum*	15	60
Cover crop subtotal		15	60
Total PLS		25	100

Table D-13: Native seed mix example for sloped areas in TDOT region III.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Side-oats grama	*Bouteloua curtipendula*	1	4
Common wood sedge	*Carex blanda*	0.88	3.5
Nodding wild rye	*Elymus canadensis*	0.88	3.5
Purple lovegrass	*Eragrostis spectabilis*	1	4
Little bluestem	*Schizachyrium scoparium*	1.25	5
Grasses, sedges, and rushes subtotal		5	20
Butterfly milkweed	*Aslepias tuberosa*	0.94	3.8
Lancelead coreopsis	*Coreopsis lanceolata*	0.75	3
Wild bergamot	*Monarda fistulosa*	0.81	3.3
Beard tongue	*Penstemon digitalis*	0.81	3.3
Gray-headed coneflower	*Ratibida pinnata*	0.81	3.5
Smooth blue aster	*Symphyotrichum leave*	0.88	3.5
Forbs subtotal		5	20
Annual ryegrass	*Lolium multiflorum*	30	60
Cover crop subtotal		30	60
Total PLS		40	100

Table D-14: Hybrid seed mix example for dry areas in TDOT region IV.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Kentucky 31 fescue	*Festuca arundinacea Schreb.*	20	26
Virginia wildrye	*Elymus virginicus*	10	13
Eastern gamagrass	*Tripsacum dactyloides*	10	13
English rye or perennial ryegrass	*Lolium perenne*	15	19.5
White clover	*Trifolium repens*	2	2.6
Red clover	*Trifolium pratense*	2	2.6
Little bluestem	*Schizachyrium scoparium*	8	10.4
Side-oats gramma	*Bouteloua curtipendula*	8	10.4
Purple lovegrass	*Eragrostis spectabilis*	2	2.6
Grasses subtotal		77	100
Annual ryegrass	*Lolium multiflorum*	30	
Cover crop subtotal		30	
Total PLS		107	100

Table D-15: Native seed mix example for wet areas in TDOT region IV.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Common wood sedge	*Carex blanda*	0.94	3.8
Yellowfruit sedge	*Carex annectens*	0.94	3.8
Fox sedge	*Carex vulpinoidea*	0.94	3.8
Common rush	*Juncus effuses*	1.25	5
Switchgrass	*Panicum virgatum*	0.94	3.8
Grasses, sedges, and rushes subtotal		5	20
Swamp milkweed	*Asclepias incarnata*	0.94	3.8
Common boneset	*Eupatorium perfoliatum*	0.75	3
Common sneezeweed	*Helenium autumnale*	0.88	3.5
Virgnia mountain mint	*Pycanthemum virginianum*	0.88	3.5
Swamp verbena	*Verbena hastata*	0.75	3
Golden zizia	*Zizia aurea*	0.81	3.3
Forbs subtotal		5	20
Annual ryegrass	*Lolium multiflorum*	15	60
Cover crop subtotal		15	60
Total PLS		25	100

Table D-16: Native seed mix example for sloped areas in TDOT region IV.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Side-oats grama	*Bouteloua curtipendula*	1	4
Common wood sedge	*Carex Blanda*	0.88	3.5
Nodding wild rye	*Elymus canadensis*	0.88	3.5
Purple lovegrass	*Eragrostis spectabilis*	1	4
Little bluestem	*Schizachyrium scoparium*	1.25	5
Grasses, sedges, and rushes subtotal		5	20
Butterfly milkweed	*Ascelpias tuberosa*	0.94	3.75
Lanceleaf coreopsis	*Coreopsis lanceolate*	0.75	3
Wild bergamont	*Monarda fistulosa*	0.81	3.25
Beard tongue	*Penstemon digitalis*	0.81	3.25
Gray-headed coneflower	*Ratibida pinnata*	0.81	3.25
Smooth blue aster	*Symohyotrichum leave*	0.88	3.5
Forbs subtotal		5	20
Annual ryegrass	*Lolium multiflorum*	30	60
Cover crop subtotal		30	60
Total PLS		40	100

Example seed mixes for solar sites, depending on general site characteristics are provided in Tables D-17 through D-19.

Table D-17: Permanent seed mix example for solar sites that is broadly applicable.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Fescue	*Festuca arundinacea*	10	29
Creeping red fescue	*Festuca rubra*	5	14
Kentucky bluegrass	*Poa pratensis*	5	14
Bermudagrass	*Cynodon dactylon*	5	14
Annual rye grass	*Lolium multiflorum*	5	14
Perennial rye grass	*Lolium perenne*	5	14
Total PLS		35	100

Table D-18: Permanent seed mix example for solar sites with dry-medium soils.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Side oats grama	*Bouteloua curtipendula*	2	24
Virginia wildrye	*Elymus virginicus*	2	24
Beaked panicgrass	*Panicum anceps/Coleataenia anceps*	0.91	11
Little barley	*Hordeum pusillum*	1	12
Little bluestem	*Schizachyrium scoparium*	2	24
River oats	*Chasmanthium latifolium*	0.37	5
	Total PLS	8.28	100

Table D-19: Permanent seed mix example for solar sites with wet-medium soils.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
American barnyard grass	*Echinochloa muricata*	2	28
Virginia wildrye	*Elymus virginicus*	2	28
Beaked panicgrass	*Panicum anceps/Coleataenia anceps*	0.87	12
Little barley	*Hordeum pusillum*	1	14
Fox sedge	*Carex vulpinoidea*	1	14
River oats	*Chasmanthium latifolium*	0.38	5
	Total PLS	7.25	100

In riparian areas, it is important to avoid invasive or threatening species that will choke out the natives. The example mixes listed for streams and stream banks avoid use of introduced species and provide a diverse list of species for wildlife habitat and disease resistance. It is acknowledged that some of the species listed may not always be available, it is up to the discretion of the designer to substitute species as needed to create a mix best suited for their site.

Table D-20: Permanent native seed mix example for streambanks across the state.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
River bank wildrye	*Elymus riparius*	0.47	7
Virginia wildrye	*Elymus virginicus*	1.15	17
River oats	*Chasmanthium latifolium*	0.21	3
American barnyard grass	*Echinochloa muricata*	0.21	3
Upland bentgrass	*Agrostis perennans*	0.04	1
Big bluestem	*Andropogon gerardii*	0.51	8
Deer tongue grass	*Panicum clandestinum*	0.3	4
Fall panicum	*Panicum anceps*	0.51	8
Switchgrass	*Panicum virgatum*	0.72	11
Fox sedge	*Carex vulpinoidea*	0.13	2
Wild senna	*Senna marilandica*	0.43	6
White snakeroot	*Eupatorium rugosum*	0.09	1
False sunflower	*Heliopsis helianthoides*	0.38	3
Bergamot	*Monarda fistulosa*	0.09	1
Cup plant	*Silphium perfoliatum*	0.43	6
Showy tickseed	*Bidens aristosa*	0.3	4
Sneezeweed	*Helenium autumnale*	0.09	1
Yellow wingstem	*Verbesina alternifolia*	0.34	5
Iron weed	*Vernonia altissima*	0.21	3
Narrow-leaved sunflower	*Helianthus angustifolius*	0.21	3
Total PLS		6.8	100

Table D-21: Permanent native seed mix example for upland areas and buffers across the state.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Virginia wildrye	*Elymus virginicus*	1.08	15
American barnyard grass	*Echinochloa muricata*	0.23	3.1
Upland bentgrass	*Agrostis perennans*	0.01	0.1
Big bluestem	*Andropogon gerardii*	0.63	8.8
Deer tongue grass	*Panicum clandestinum*	0.63	8.8
Fall panicum	*Panicum anceps*	0.63	8.8
Switchgrass	*Panicum virgatum*	1.08	15
Fox sedge	*Carex vulpinoidea*	0.22	3
Wild senna	*Senna marilandica*	0.45	6.3
Illinois bundleflower	*Desmanthus illinoensis*	0.23	3.1
False sunflower	*Heliopsis helianthoides*	0.27	3.8
Spiked blazing star	*Liatris spicata*	0.23	3.1
Bergamot	*Monarda fistulosa*	0.05	0.6
Cup plant	*Silphium perfoliatum*	0.45	6.3
Showy tickseed	*Bidens aristosa*	0.27	3.8
Joe-pye weed	*Eupatorium fistulosum*	0.09	1.3
Sneezeweed	*Helenium autumnale*	0.09	1.3
Yellow wingstem	*Verbesina alternifolia*	0.23	3.1
Iron weed	*Vernonia altissima*	0.18	2.5
Narrow-leaved sunflower	*Helianthus angustifolius*	0.18	2.5
Total PLS		7.2	100

Table D-22: Permanent native seed mix example for wetlands across the state.

Common Name	Botanical Name	PLS (lbs/ac)	PLS by Weight (%)
Fowl manna grass	*Glyceria striata*	0.21	5
Hop sedge	*Carex lupulina*	0.55	14
Rufous bulrush	*Scirpus pendulus*	0.05	1
Creeping spike rush	*Eleocharis palustris*	0.21	5
Frank's sedge	*Carex frankii*	0.31	8
Eastern bur reed	*Sparganium americanum*	0.55	14
Nodding sedge	*Carex crinita*	0.25	6
Green bulrush	*Scirpus atrovirens*	0.03	1
Soft rush	*Juncus effusus*	0.09	2
Leathery rush	*Juncus coriaceus*	0.02	0
Rice cut grass	*Leersia oryzoides*	0.25	6
Button bush	*Cephalanthus occidentalis*	0.23	6
Pennsylvania smartweed	*Polygonum pensylvanicum*	0.25	6
Blue flag	*Iris virginica*	0.23	6
Rosemallow	*Hibiscus moscheutos*	0.16	4
Seed box	*Ludwigia alternifolia*	0.06	1
Monkey flower	*Mimulus ringens*	0.06	1
Arrowhead	*Sagittaria latifolia*	0.19	5
Swamp milkweed	*Asclepias incarnata*	0.33	8
Total PLS		4	100

Restricted Seeds

Seed sold or produced in Tennessee is subject to regulatory oversight under the Tennessee Department of Agriculture. Specifically, Rules Chapter 0080-05-06 outlines a list of prohibited noxious weeds that may not be included in seed mixes, as well as a restricted list of noxious weeds within produced seeds. These regulations are intended to control the presence of noxious weed impurities in produced and distributed seed.

Invasive Species

Invasive species are defined by two key characteristics: they are non-native to the ecosystems in which they are found, and their presence causes, or is likely to cause, harm to the environment, economy, infrastructure, or human health. When not actively managed, these species can displace native flora and fauna, degrade biodiversity, disrupt ecosystem services, compromise water and soil resources, and negatively impact agriculture, forestry, recreation, cultural assets, and public safety (USDA, a).

The purpose of this section is to highlight non-native plant species in Tennessee that are invasive or pose a high risk of becoming invasive. Though not regulatory, they provide critical guidance to landowners and resource managers on making informed, responsible plant choices. The Tennessee Invasive Plant Council (TN-IPC) acknowledges that not all introduced plants are harmful, but those that naturalize and spread can have serious ecological consequences. As such, the TN-IPC identifies species that are more likely to cause such damages and strongly advises against their use. Additional resources for identifying invasive species in forests, grasslands, wetlands, and riparian areas include USDA (b): *High Priority Nonnative Invasive Species of Southern Forest and Grassland Ecosystems* and USDA (c): *Assessment of Nonnative Invasive Species* both of which offer region-specific lists and visual references for early detection and prevention.

Vegetation selection for erosion control must strike a balance between stabilization performance and ecological compatibility. Commonly used grasses such as tall fescue and bermudagrass are highly effective at controlling soil loss due to their dense, mat-forming growth, yet offer limited habitat value for wildlife and can suppress native species. For this reason, plant choices should be tailored to site-specific conditions and long-term project goals. If using species listed in the invasive species tables (Tables D-22 and D-23), proceed with caution. Table D-22 identifies species known to invade intact native ecosystems and threaten community integrity and restoration efforts, while Table D-23 focuses on species problematic in disturbed environments, where they exhibit aggressive traits even outside of pristine areas.

Table D-23: Invasive grass, forbs, and herb species in natural areas.

Common Name	Botanical Name	Threat Status	Category
Centaurea repens (L.), Russian knapweed	*Centaurea repens* (L.) DC.	Emerging	Forb/herb
Garlic mustard	*Alliaria petiolata* (Bieb.) Cavara & Grande	Established	Forb/herb
Alligatorweed	*Alternanthera philoxeroides* (Mart.) Griseb.	Established	Forb/herb
Hairy iointgrass, small carpetgrass	*Arthraxon hispidus* (Thunb.) Makino	Established	Grass
Elephant grass, giant reed	*Arundo donax* L.	Emerging	Grass
Hungarian brome, smooth brome	*Bromus inermis* Leyss.	Established	Grass
Centaurea biebersteinii DC., spotted knapweed	*Centaurea stoebe* L. ssp. *micranthos* (S.G. Gmel. ex Gugler) Hayek	Established	Forb/herb
Fleeceflower, Japanese knotweed, Mexican bamboo, polygonum cuspidatum seib. and zucc.	*Fallopia japonica* (Houtt.) Ronse Decr.	Established	Forb/herb
Giant cow parsnip, giant hogweed	*Heracleum mantegazzianum* Sommier & Levier	Emerging	Forb/herb
Cogongrass	*Imperata cylindrica* (L.) P. Beauv.	Emerging	Grass
Chinese lespedeza, sericea lespedeza	*Lespedeza cuneata* (Dum.-Cours.) G. Don	Established	Forb/herb
Creeping lilyturf, creeping liriope, lilyturf, monkey-grass, big blue lilyturf	*Liriope spicata* (Thunb.) Lour.	Emerging	Forb/herb
Japanese climbing fern	*Lygodium japonicum* (Thunb. ex Murr.) Swartz	Emerging	Forb/herb
Japanese stiltgrass, Nepalese browntop, nepalgrass	*Microstegium vimineum* (Trin.) A. Camus	Established	Grass
Chinese silver grass, eulalia grass, maiden grass, zebra grass	*Miscanthus sinensis* Anderss.	Established	Grass

Asian spiderwort, marsh dayflower	*Murdannia keisak* (Hassk.) Hand.-Maz.	Established	Forb/herb
Beefsteak plant, Chinese basil, perilla, Perilla mint	*Perilla frutescens* (L.) Britton	Established	Forb/herb
Asiatic tearthumb, mile-a-minute weed	*Persicaria perfoliata* (L.) H. Gross	Emerging	Forb/herb
Common reed	*Phragmites australis* (Cav.) Trin. ex Steud.	Established	Grass
Golden bamboo	*Phyllostachys aurea* Carr. ex A. & C. Rivere	Emerging	Grass
Ficaria verna huds., fig buttercup, lesser celandine	*Ranunculus ficaria* L.	Emerging	Forb/herb
Itchgrass	*Rottboellia cochinchinensis* (Lour.) W.D. Clayton	Emerging	Grass
Johnson grass	*Sorghum halepense* (L.) Pers.	Established	Grass
PunctureVine	*Tribulus terrestris* L.	Emerging	Forb/herb
Coltsfoot	*Tussilago farfara* L.	Established	Forb/herb

Table D-24: Invasive grass, forbs, and herb species in disturbed areas.

Common Name	Botanical Name	Category
Japanese chaff flower	*Achyranthes japonica* var. *hachiijoen*	Forb/herb
Creeping bentgrass	*Agrostis stolonifera* L.	Grass
Field garlic	*Allium vineale* L.	Forb/herb
Sessile joyweed	*Alternanthera sessilis* (L.) R. Br. ex DC.	Forb/herb
Field brome, Japanese bromegrass	*Bromus japonicus* Thunb. ex Murr.	Grass
Rye Brome	*Bromus secalinus* L.	Grass
Barren bromegrass, poverty brome	*Bromus sterilis* L.	Grass
Cheat grass, downy brome, downy chess	*Bromus tectorum* L.	Grass
Corn gromwell, field gromwell, bastard alkanet, stoneseed	*Buglossides arvensis* (L.) I.M. Johnst.	Forb/herb
Hare's ear, hound's ear, thorough-wax (thorow-wax)	*Bupleurum rotundifolium* L.	Forb/herb
Bristly thistle, curled thistle, plumeless thistle, spiny plumeless thistle	*Carduus acanthoides* L.	Forb/herb
Musk thistle, nodding thistle	*Carduus nutans* L.	Forb/herb
Bachelor's button, garden cornflower, bluebottle, raggedy sailors	*Centaurea cyanus* L.	Forb/herb
Chicory, coffeeweed, succory, blue sailors	*Cichorium intybus* L.	Forb/herb
Canada thistle	*Cirsium arvense* (L.) Scop.	Forb/herb
Bull thistle, common thistle, spear thistle	*Cirsium vulgare* (Savi) Ten.	Forb/herb
Poison hemlock	*Conium maculatum* L.	Forb/herb
Queen Anne's lace, wild carrot	*Daucus carota* L.	Forb/herb
Common teasel, fuller's teasel	*Dipsacus fullonum* L.	Forb/herb
Weeping lovegrass or African lovegrass	*Eragrostis curvula* (Schrad.) Nees	Grass

Firewheel, Indian blanket, rose-ring gaillardia	*Gaillardia pulchella* Foug.	Forb/herb
Texas blueweed	*Helianthus ciliaris* DC.	Forb/herb
Dame's rocket	*Hesperis matronalis* L.	Forb/herb
Yellow flag iris	*Iris pseudacorus* L.	Forb/herb
Korean clover, Korean lespedeza	*Kummerowia stipulacea* (Maxim.) Makino	Forb/herb
Japanese clover	*Kummerowia striata* (Thunb.) Schindl.	Forb/herb
Field pepperweed	*Lepidium campestre* (L.) W.T. Aiton	Forb/herb
Ox-eye daisy	*Leucanthemum vulgare* Lam.	Forb/herb
Tall fescue	*Lolium arundinaceum* (Schreb.) Darbysh.	Grass
Bird's-foot trefoil, birdfoot deervetch, bloomfell, cat's clover, crowtoes, ground honeysuckle	*Lotus corniculatus* L.	Forb/herb
Creeping Jenny, moneywort	*Lysimachia nummularia* L.	Forb/herb
White sweet clover	*Melilotus alba* Medik.	Forb/herb
Yellow sweet clover	*Melilotus officinalis* (L.) Lam.	Forb/herb
Spearmint	*Mentha spicata* L.	Forb/herb
Peppermint	*Mentha* × *piperita* L.	Forb/herb
Grape hyacinth	*Muscari neglectum* Guss. ex Ten.	Forb/herb
True forget-me-not, yelloweye forget-me-not, water scorpion-grass forget-me-not	*Myosotis scorpioides* L.	Forb/herb
Star of Bethlehem	*Ornithogalum umbellatum* L.	Forb/herb
Wild parsnip	*Pastinaca sativa* L.	Forb/herb
Annual canary grass	*Phalaris canariensis* L.	Grass
Bristly lady's thumb, bunchy knotweed, long-bristled smartweed, oriental lady's thumb	*Polygonum caespitosum* var. *longisetum*	Forb/herb

Spotted ladysthumb	*Polygonum persicaria* L.	Forb/herb
St. Anthony's turnip	*Ranunculus bulbosus* L.	Forb/herb
Prickly Russian thistle	*Salsola tragus* L.	Forb/herb
Java-bean, sicklepod senna	*Senna obtusifolia* (L.) H.S. Irwin & Barneby	Forb/herb
Chinese foxtail, nodding foxtail-grass	*Setaria faberi* R.A.W. Herrm.	Grass
Foxtail millet	*Setaria italica* (L.) P. Beauv.	Grass
Smooth millet, yellow foxtail	*Setaria pumila* (Poir.) Roem. & Schult.	Grass
Giant green foxtail, green bristlegrass, green foxtail, green millet	*Setaria viridis* (L.) P. Beauv.	Grass
Perennial sowthistle, sowthistle	*Sonchus arvensis* L.	Forb/herb
Yellow goat's-beard, yellow salsify	*Tragopogon dubius* Scop.	Forb/herb
Common mullein	*Verbascum thapsus* L.	Forb/herb

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Seed Mix Appendix Committee Participants:

Todd Crabtree	TDEC-Natural Heritage
Adam Dattilo	TDEC-Natural Heritage
Lauren Deem	Profile Products LLC
Evelyn Diorio	Tennessee Department of Transportation
Becky Grubbs-Bowling	University of Tennessee Knoxville
Jeremy Hamlington	Roundstone Native Seed LLC
Chip Hannah	Tennessee Department of Transportation
Nathaniel Hicklin	NRCS
Robert Hoffman	Roundstone Native Seed LLC
John L. Seymour	Roundstone Native Seed LLC
Andrea Ludwig	University of Tennessee Knoxville
Michael McCord	TWRA
Mike Ross	University of Tennessee Knoxville

Marc S. Theisen	Profile Products LLC
Eric Shiflett	TDEC- Division of Water Resources
Joe Stephens	Stephens Supply Co.
Virginia Sykes	University of Tennessee Knoxville
Trent Thomas	Tennessee Department of Transportation
Tyler Wayland	Roundstone Native Seed LLC
Shawn Wurst	Tennessee Department of Transportation

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Appendix E - Definitions

Bank	**Bank** (e.g., stream bank or river bank) is the rising ground bordering the channel of a jurisdictional water (adapted from EPA CGP)
Biodegradable	**Biodegradable** means an item is capable of decomposing under ambient soil conditions into naturally occurring materials over a period of time (e.g., one year). (EPA CGP)
Construction Waste	**Construction Waste** is discarded material (such as packaging materials; scrap construction materials; masonry products; timber, steel, pipe, and electrical cuttings; plastics; and styrofoam). (EPA CGP)
Conveyance Channel	**Conveyance Channel** is a temporary or permanent waterway designed and installed to safely convey stormwater flow within and out of a construction site. (EPA CGP)
Dewatering	**Dewatering** is the act of draining accumulated stormwater and/or ground water from building foundations, vaults, and trenches, or other similar points of accumulation. (EPA CGP)
Discharge Point	**Discharge Point** is the location where collected and concentrated stormwater flows or dewatering water are discharged from the construction site. (EPA CGP)
Drainageway	**Drainageway** is an open linear depression, whether constructed or natural, that functions for the collection and drainage of surface water. (EPA CGP)
Installation	**Install or Installation**, when used in connection with stormwater controls, is to connect or set in position stormwater controls to make them operational. (EPA CGP)
Native Topsoil	**Native Topsoil** is the uppermost layer of naturally occurring soil for a particular area, and is often rich in organic matter, biological activity, and nutrients. (EPA CGP)

Water of the United States	**Receiving Water** is the jurisdictional water into which the regulated stormwater discharges. (adapted from EPA CGP)
Run-On	**Run-On** is a source of stormwater that drains from land located upslope or upstream from the regulated site in question. (adapted from EPA CGP)
Snowmelt	**Snowmelt** is the conversion of snow into overland stormwater and ground water flow as a result of warmer temperatures. (EPA CGP)
Spill	**Spill** is the release of a hazardous or toxic substance from its container or containment. (EPA CGP)
Stormwater Control	**Stormwater Control** refers to any best management practice or other method (including narrative effluent limitations) used to prevent or reduce the discharge of pollutants to waters of the United States. (EPA CGP)
Stormwater Inlet	**Stormwater Inlet** is a structure placed below grade to conduct water used to collect stormwater runoff for conveyance purposes. (EPA CGP)
Storm Event	**Storm Event** is a precipitation event that results in a measurable amount of precipitation. (EPA CGP)
Storm Sewer	**Storm Sewer** is a system of pipes (separate from sanitary sewers) that carries stormwater runoff from buildings and land surfaces. (EPA CGP)
Upland	**Upland** is the dry land area above and 'landward' of the ordinary high water mark. (EPA CGP)
Visual Turbidity	**Visual Turbidity** is present when there is a sediment plume in the discharge or the discharge appears cloudy, opaque, or has a visible contrast that can be visually identified by an observer. (EPA CGP)