Wetlands Engineering Handbook

Section 5

Establishing Proper Hydrologic Conditions

Section 5
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5-1 Overview of Hydrologic and Hydraulic Analyses[1]

Background

The long-term success of a wetland restoration project is dependent upon a viable water source and supportive hydrology. Surface water can enter a wetland through precipitation, runoff, streamflow, incoming tides; and spillover from an adjacent water body. Wetlands may also receive water from groundwater and natural springs. Water entering the wetland is continuously exchanged between the surface wetland, the groundwater, and the receiving waters. Water exits the wetland through natural drainage channels, natural groundwater gradients, seepage, evapotranspiration, outgoing tides, and water control structures. Regional rainfall often follows a seasonal pattern which can be evaluated and applied to the design of the wetland project. Other hydrologic features such as infiltration, seepage, and evapotranspiration are controlled by large-scale geomorphic features, soil type, terrain, and geographic location. These features may lend themselves to local modification to accommodate the wetland project.

Wetland hydraulic design is an iterative procedure, consisting of 1) proposed hydraulic design, 2) site drainage analysis and surface flow hydrodynamic analysis with the proposed features in place, 3) evaluation of the proposed design against the specified design criteria, and 4) modification of the proposed design. Designs must meet the design criteria and be compatible with site limitations. The project budget can influence the project design because maintenance requirements, initial investment, and annual costs will be limited by the funding available.

Scope

This section provides an overview of the appropriate tools for the analysis, design, and construction of wetland hydraulic features. For an overview of wetland hydrology, please refer to Chapter 2-4. Chapter 5-2 describes surface and subsurface drainage analyses and surface flow hydrodynamic analyses. An overview of the design of water control structures is given in Chapter 5-3. Chapter 5-4 describes methods for erosion control and shoreline protection for wetland sites.

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[1] By Lisa C. Roig

Hydrologic and Hydraulic Design Process

Hydrologic design and analysis procedures principally involve a description of the temporal and spatial distributions of rainfall, runoff, water table fluctuations, and tides. The hydrologic processes affecting wetlands include direct precipitation, runoff, streamflow, infiltration, surface storage, subsurface storage, and groundwater flow. The circulation of surface water within the wetland, also called surface water hydrodynamics, can also be important for the purposes of wetland design and engineering depending upon the rate of water exchange, the currents within the wetland itself, and the design criteria. The types of design features most often considered for wetlands are water control structures, site grading, dredged channels, diversion structures, culverts and erosion control measures. The hydrologic analyses required to design these features may include one or more of the following: 1) determining the design storm or design tide; 2) determining the maximum and minimum water surface elevations; 3) determining the circulation patterns and maximum velocities; 4) determining the propagation of wind waves; 5) determining the extent and duration of inundation events; 6) determining the minimum required inflows; or 7) determining the maximum permissible outflows.

The hydrologic analyses required for the design of a wetland project are normally divided into two phases: 1) baseline hydrologic analysis of the site; and 2) hydrologic/hydraulic design and analysis. Baseline hydrologic analysis of the site is required before developing the preliminary designs and is more in depth than what would be produced during site selection. It provides information about the sources of water, their magnitudes and the current distribution of water on the site. A flow chart for developing a baseline hydrologic analysis of the site is provided in Figure 5-1. The baseline hydrologic analysis may also provide information during the hydrologic-hydraulic design process, but additional analyses will be required depending upon the feature to be designed. A generalized flow chart of the site design process is provided in Figure 5-2.

Hydrologic and Hydraulic Analyses

Hydrologic and hydraulic analyses refer to the process of gathering, organizing, and applying hydrologic data. The purposes of these analyses are to understand the distribution, abundance, and dynamic behavior of water on the site. Varying degrees of hydrologic analyses are required during initial site assessment, during site selection, during the development of conceptual designs, and during the design of the engineering works. Hydrologic analyses that describe the relationship of the wetland site to the contributing watershed usually need to be done only once. The modification of onsite hydrology and its affect on the watershed downstream of the wetland project must be repeated for each design and for each set of seasonal hydrologic conditions specified in the design criterion. Table 5-1 lists the most common hydrologic analysis methods and describes their intended use. The data requirements for each type of hydrologic analysis are presented in Table 5-2.

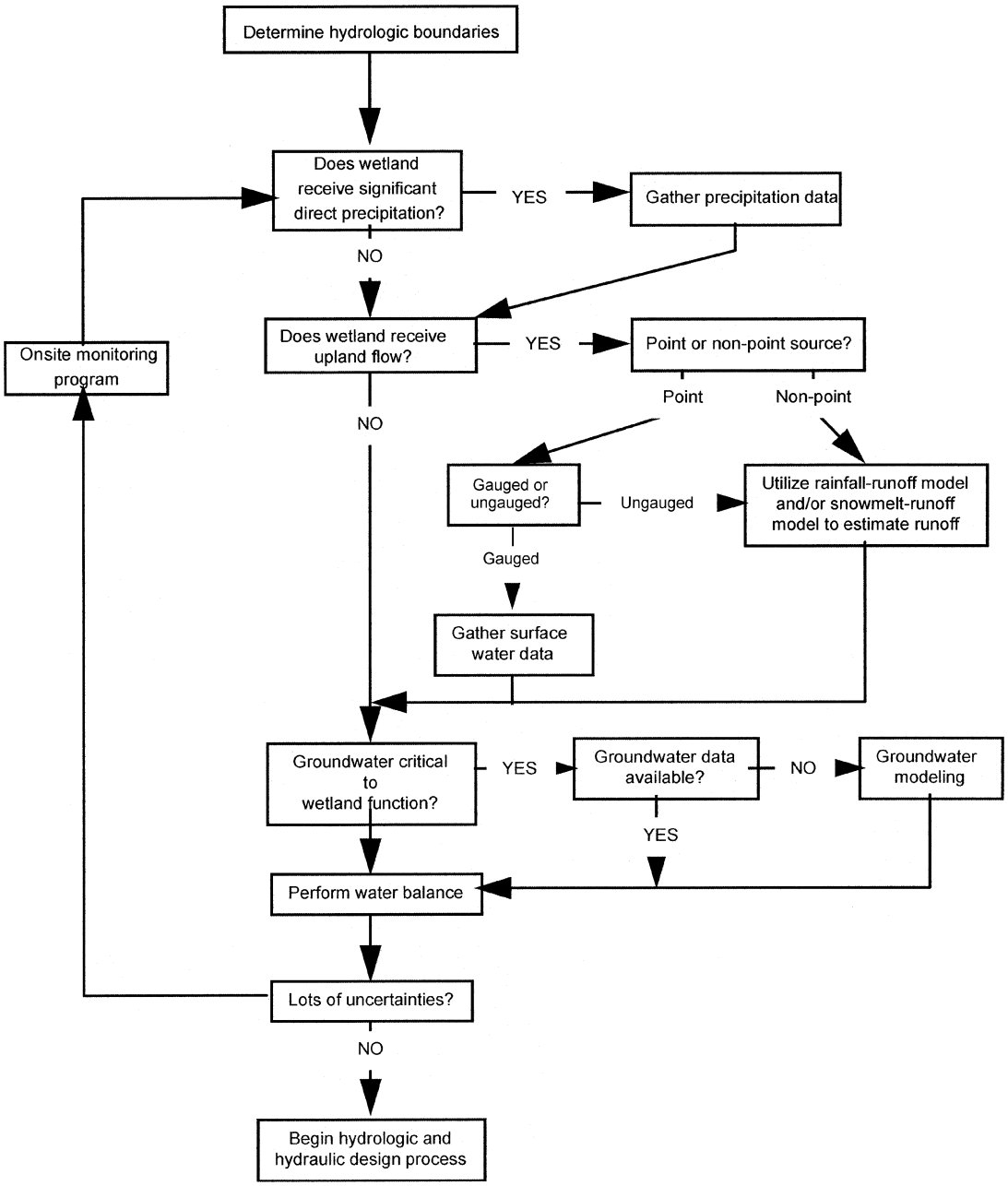

Figure 5-1. Baseline hydrologic analysis for wetlands.

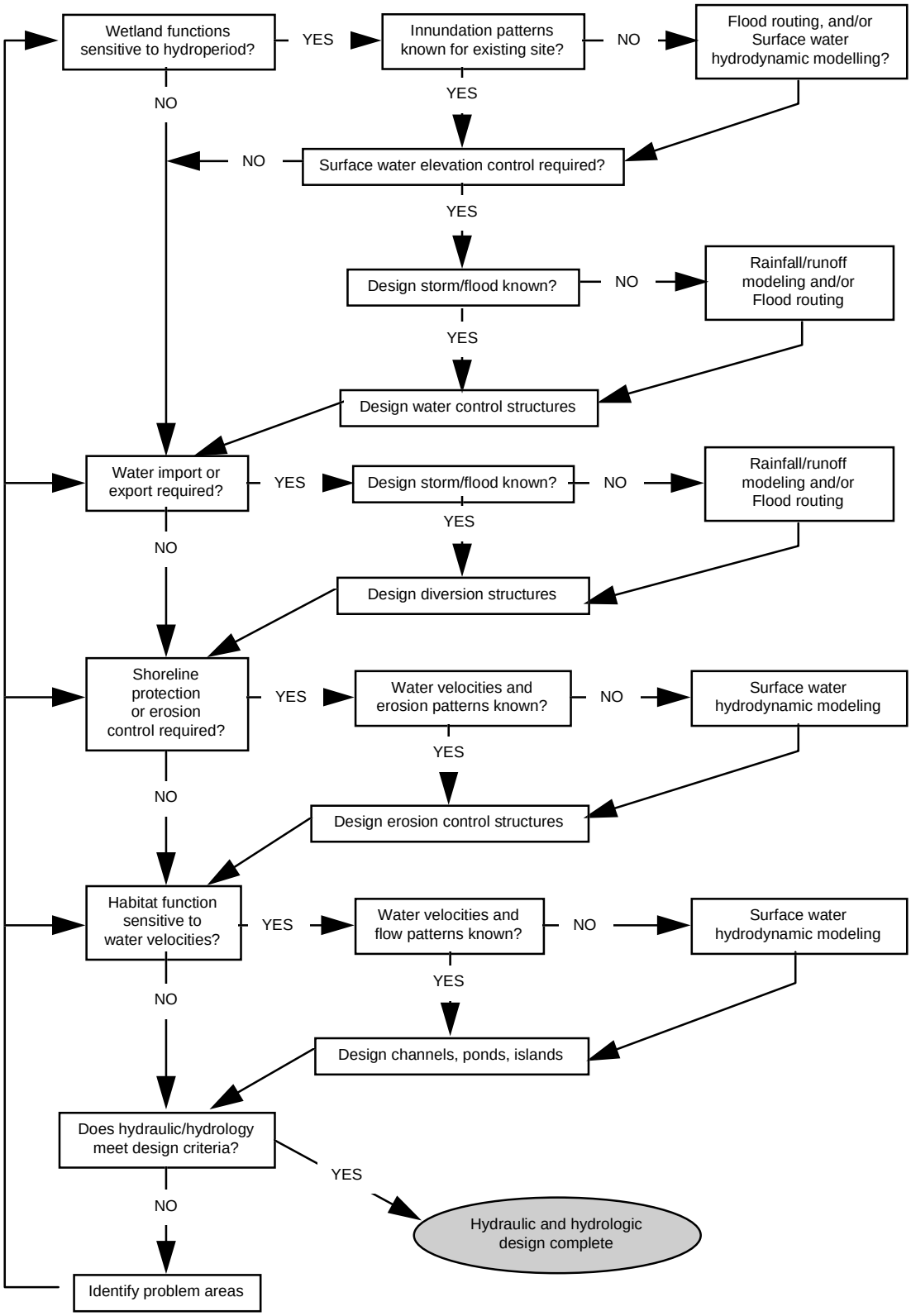

Figure 5-2. Hydrologic and hydraulic design process for wetlands.

Table 5-1
Types of Hydrologic Analysis Tools

Hydrologic Analysis Tool	Purposes
Water Balance	Quantify evaporation or seepage. Identify all water sources and sinks within the hydrologic unit. Determine adequacy of supply.
Rainfall-Runoff Models	Quantify the amount of runoff generated by a rainfall event. Determine the peak discharge at the outlet per storm event. Determine the time to peak discharge at the outlet per storm event. Determine the recession time per storm event.
Snowmelt-Runoff Models	Quantify the amount of runoff generated by a snowmelt event. Quantify the amount of runoff generated by a snowmelt season.
Frequency Analysis	Determine recurrence interval of a storm event. Determine reliability of a design structure.
Flood Routing	Predict temporal and spatial movement of a flood wave in a channel network.
Groundwater Flow Models	Predict temporal and spatial distribution of groundwater in one-, two-, or three-dimensions.
Surface Water Hydrodynamic Models	Predict temporal and spatial distribution of surface water in one-, two-, or three-dimensions.

The following project attributes should be considered when selecting a hydrologic analysis tool: 1) the demands of the stated wetland design criteria; 2) the type of control structures or construction features to be designed; and 3) the project cost. These attributes are discussed briefly below.

Demands of the Stated Wetland Design Criteria

Table 5-3 lists a few example hydrologic design criteria and their corresponding hydrologic analysis requirements. Note that the example design features listed are only a sample of the broad range of possible solutions. The appropriate design choice for a particular site will depend upon the site configuration and the results of the pre-design hydrologic analysis.

Structural Features

Certain structural features require a specific hydrologic analysis to determine the design parameters. The most common type of analysis is the determination of a design storm or design tide. The design storm is the maximum storm event that an engineering feature can accommodate without failure. In wetland restoration, the discharge resulting from a design storm is used to determine the size of the outlet structure, the basin storage capacity, the minimum culvert

Table 5-2
Data Requirements for Hydrologic Analysis

Hydrologic Analysis Tool	Data Required	Desired Period of Record	Desired Frequency
Water Balance	Precipitation	≥ 1 year	daily
	Streamflows	≥ 1 year	daily
	Groundwater storage	≥ 1 year	weekly
	Surface storage	≥ 1 year	daily
	Point sources and withdrawals	≥ 1 year	daily
	Land use and vegetation cover	≥ 1 year	seasonal
	Watershed characteristics	once	-
Rainfall-Runoff Models	Precipitation	storm duration	depends upon basin response time
	Streamflows	depends upon basin response time	depends upon basin response time
	Watershed characteristics	once	once
	Land use and vegetation cover	≥ 1 year	seasonal
	Antecedent soil moisture	once prior to storm event	once
Snowmelt-Runoff Models	Average air temperature	melting season	daily
	Snow cover	" "	" "
	Snow water content	" "	" "
	Snow surface albedo	" "	" "
	Solar radiation	" "	" "
	Precipitation (rain and snow)	" "	" "
Frequency Analysis	Precipitation time series	≥ 1 year	depends upon basin response time
	Streamflow time series	≥ 1 year	depends upon basin response time
	Tide height time series	annual	at least hourly
Flood Routing	Basin inflow time series	duration of storm event	depends upon basin response time
	Channel cross-section profile, reach length, and bed slope for all channel segments	once for stable channels	once for stable channels
	Channel bed roughness for all channel segments	once for stable, un-vegetated channels	once for stable, un-vegetated channels
Groundwater Flow Models	Aquifer stratigraphy and hydraulic conductivity of each layer	once	once
	Initial piezometric surface throughout the basin	once per simulation event	once per simulation event
	Subsurface flow boundary conditions	once per simulation event	once per simulation even
	Sources and sinks of groundwater	duration of simulation event	depends upon basin response time
Surface Water Hydrodynamic Models	Basin bathymetry and distributed bed roughness	once for stable, un-vegetated channels	once for stable, un-vegetated channel
	Initial water surface elevation and velocity field	once per simulation event	once per simulation event
	Surface flow boundary conditions	once per simulation event	once per simulation event
	Sources and sinks of surface water	duration of simulation event	depends upon basin response time

Table 5-3
Hydrologic Analyses Required to Meet Example Design Criteria

Example Design Criteria	Type of Hydrologic Analyses Required	Example Design Features
Pass design flood without exceeding maximum specified water surface elevation in the wetland.	1. Determine the design rainfall event. 2. Quantify the runoff from the design event. 3. Route the storm hydrograph through the wetland. 4. Determine the maximum discharge at the control point. 5. Design the structure to meet the design criteria.	Overflow structures, culverts, drop inlet structures, gates, valves, plugs, pumps
Determine maximum water surface elevation during design flood.	1. Determine the design rainfall event. 2. Quantify the runoff from the design event. 3. Route the storm hydrograph through the wetland. 4. Determine the maximum water surface elevation.	Levees, overflow structures, access structures
Maintain water level within a specified range.	1. Determine the design flood and design drought events. 2. Propose several possible designs as dictated by the site configuration and pre-design hydrologic analyses. 3. Quantify the surface flow abundance and distribution under both flood and drought conditions for each proposed design. 4. Compare the water levels achieved under proposed designs against the design criteria. 5. Refine the best designs and repeat the analyses.	Overflow structures, culverts, weirs, gates, valves, plugs, pumps
Do not exceed a specified water velocity in the wetland.	1. Determine the design flood event. 2. Propose several possible designs as dictated by the site configuration and pre-design hydrologic analyses. 3. Predict surface flow hydrodynamics for each proposed design. 4. Compare the velocities achieved under proposed designs against the design criteria. 5. Refine the best designs and repeat the analyses.	Land surface grading and channel dredging, culverts, inlet and outlet structures
Reduce the flood peak of the 100 year storm by 20 percent.	1. Determine the 100-year storm event. 2. Route the 100-year storm through existing wetland or channel network. 3. Determine the peak discharge. 4. Propose several possible designs. 5. Route the 100-year storm through proposed channel network. 6. Select the designs that meet the design criteria. 7. Refine the best designs and repeat the analyses.	Land surface grading and channel dredging, culverts, inlet and outlet structures,
Increase groundwater recharge at the restoration site by 20 percent	1. Determine the infiltration rates, seepage rates, temporal and spatial distribution of groundwater at existing site. 2. Propose several designs with different surface water detention and storage capacities. 3. Predict the recharge and distribution of groundwater under each of the proposed plans. 4. Select the designs that meet the design criteria. 5. Refine the best designs and repeat the analyses.	Land surface grading, outlet structures, valves, gates, plugs

dimensions, and/or the minimum channel conveyance. Similarly, the design tide is the tide with the maximum water surface elevation that can be accommodated by the engineering project. The design storm and the design tide are normally determined by means of frequency analysis.

The peak discharge resulting from a design storm is a useful piece of information for the design of a culvert or gate. If the watershed is gauged, the time series of discharge can be used to determine the peak flow. Unfortunately wetland restoration sites are frequently located in ungauged local depressions, floodplains and side channels. These small watersheds derive inflow from local drainage and urban runoff which may be supplemented by imported water. The peak flow in an ungauged watershed is determined by computing the runoff generated by the design storm.

Tidal data from a nearby harbor or marina can sometimes be used to determine the design tide when onsite data are unavailable. The peak tide height from the design tide will determine the height of levees, islands, and weirs. The tidal wave height may be either dampened or amplified as it passes through the inlet depending upon the inlet morphology. For large restoration sites, complex inlet morphologies, and ungauged tidal inlets, an onsite tidal gauge should be installed to determine the tidal phase and amplitude in relation to permanent local monitoring stations. If no local data are available a hydrodynamic model may be necessary to determine local tide levels.

Project Cost

The overall cost of the project will bear upon the extent of the hydrologic analyses. Water balance, rainfall-runoff models, snowmelt-runoff models, and frequency analyses are low-cost procedures in terms of engineering labor. They are also less accurate than the more sophisticated modeling techniques. Hydraulic flood routing, groundwater modeling, and surface water hydrodynamic modeling can be expensive and time-consuming procedures. These more accurate and expensive analyses are well justified when the wetland restoration project involves extensive structural modification to the site. Because the use of simulation models reduces the uncertainty in the design parameters, a more efficient design structure can result. Appropriate use of models during the design phase could save millions of dollars in construction and material costs on large projects. Less accurate and less costly analyses are sufficient when the project requires only minor engineering works. However a higher factor of safety should be applied.

5-2 Surface and Subsurface Drainage Analysis[1]

Surface Water Analysis Methods

The following paragraphs describe the types of standard hydrologic analyses used to interpret hydrologic data and make it useful for engineering purposes. The specifics of each of these analyses are described in standard hydrology texts, and so are not repeated here. Useful references include Maidment (1993), Bedient and Huber (1992), Viessman et al. (1977), and Hydrologic Engineering Center (1980). These methods include statistical data analyses, simple models, and empirical formulas.

Hydrologic Data

Hydrologic records that are of value for wetland design include precipitation, wind, temperature, streamflows, lake levels, and river stages. Streamflow data are collected by various state and federal agencies, including the U. S. Geological Survey (USGS), U.S. Army Corps of Engineers (USACE), state departments of water resources, irrigation districts, and municipalities. The USGS serves as a clearinghouse for data from most of these sources, and the annual streamflow records for every station are published by state in Surface Water Supply of the United States, a USGS Water Supply Paper Series. Lake levels and river stages are often monitored by the USACE or the U. S. Bureau of Reclamation (USBR) in the vicinity of federally maintained dams, reservoirs, and levee systems. River stages are also monitored by state water resource agencies and municipalities that maintain any type of flood control project. Tide gauges in estuarine systems are operated by the USGS, and by the National Oceanographic and Atmospheric Agency (NOAA) in coastal areas. Precipitation data are collected by the National Weather Service, by local weather bureaus, and by state resource agencies. Contact the local office of the National Weather Service for more information.

Water Balance

One meaningful way to organize data is to prepare a water balance. A water balance is a systematic method for quantifying the hydrologic components that are important within a

[1] By Lisa C. Roig

specified drainage unit. A water balance includes all of the major sources and sinks of water within the hydrologic boundaries of the system. It is a useful tool for identifying water supply problems, for identifying preliminary design opportunities, and for assessing certain impacts of proposed engineering measures. A water balance is often used to estimate the magnitude of unknown hydrologic components such as groundwater flow and infiltration losses. The Hydrologic Engineering Center (1980) has published a guide for the preparation of water balances. The procedure published therein is summarized below:

1. Identify the water supply components, including precipitation, streamflow, surface storage, groundwater pumpage, groundwater storage, imported water, and return flow.

2. Identify the water use components, including withdrawals, water rights, instream flow requirements, evaporation, evapotranspiration, and seepage.

3. Select the water balance boundaries which may be hydrologic boundaries, institutional boundaries, or a combination thereof.

4. Select the period of analysis which may be a historic drought period, a recent past period, a future period, a climatic year, a water year, or a calendar year.

5. Select the level of temporal and spatial resolution.

6. Write the water balance equation which can have several forms.

    a. For flow through systems, the water balance equation can be written in terms of flow rates, i.e., Downstream Flow = Upstream Flow + Local Inflow - Depletions - Withdrawals

    b. For surface storage systems, the water balance equation can be written in terms of volumes, i.e., Surface Storage Remaining = Inflow + Storage - Depletions - Withdrawals

    c. For groundwater storage systems, the water balance equation can be written in terms of volumetric change, i.e., Change in Storage = Recharge - Pumpage + Inflow - Outflow

7. Quantify the water balance components. Guidance for this important step is provided in Hydrologic Engineering Center (1980).

8. Interpret the results.

Two examples of water budget calculations applied to wetlands are Gilvear et al. (1993) and Vardavas (1989).

Frequency Analysis

Frequency analysis uses the historic time series of measured rainfall, runoff, or tide height at a gauge to determine the probability of recurrence of a given event. Continuous data taken at a

gauging station are quantized and tabulated in order to determine the frequency of occurrence of a particular flow rate or precipitation rate. The probability of occurrence of each quantized class of measurements is then computed according to the cumulative frequency distribution. The design event is chosen as the event having a probability of recurrence that is less than the threshold of risk prescribed by project objectives. Methods of computing the probability density function for hydrologic time series are treated extensively in the literature. A comprehensive treatment of the subject is provided by Bedient and Huber (1992) and in many other standard hydrology texts.

Rainfall-Runoff Analysis

Rainfall-runoff prediction is extremely important for wetland hydrologic design. Most wetland projects are constructed in watersheds too small for historical streamflow data to be available. Several methods have been developed to estimate the storm hydrograph characteristics for watersheds that are not continuously monitored. The characteristics of the wetland hydrograph that may be important for the design of control structures are the peak discharge, the time to peak, and the recession curve. In the following paragraphs, simple hydrologic analysis techniques are described that can provide ballpark estimates of the runoff in ungauged watersheds. The limitations of each of these methods are described. It is recommended that simple techniques be used during pre-design evaluation of the site and as a first analysis tool during the design phase. During the course of the analysis, the engineer is likely to discover that the assumptions associated with the simple analysis result in a larger margin of error than is acceptable for design purposes. More accurate hydraulic routing and hydrodynamic modeling techniques may be required to completely describe runoff generation, distribution, and abundance in ungauged watersheds.

Rational Method

The simplest rainfall-runoff method is the rational method. The rational method can be stated as

$$Q_{peak} = \sum_{j=1}^{n} C_j i A_j \tag{5-1}$$

where

Q_{peak} = peak discharge at the outlet (L^3/T)
j = index of catchment subregions
n = number of catchment subregions upstream of the outlet
C_j = runoff coefficient corresponding to the predominant land use or vegetation type in subregion j
i = rainfall intensity (L/T)
A_j = surface area of subregion j (L^2)

While the rational method is simple to apply, some of the implicit assumptions limit its application:

a. Rain falls at a uniform intensity over the catchment area

b. The duration of rainfall is sufficient that the entire catchment area contributes to the discharge at the outlet (equilibrium hydrograph).

c. Storage and infiltration losses are proportional to the rainfall intensity, and are independent of antecedent precipitation. The rational method is often used in engineering because the peak discharge computed by this method is the equilibrium discharge, and is therefore greater than the discharge expected from an actual storm of limited duration.

In wetland hydraulic design, the specification of runoff coefficients is particularly important. The runoff coefficient accounts for depression storage, infiltration, and evapotranspiration of rainfall. If the wetland is completely submerged during a storm event then depression storage and infiltration can be ignored. The peak discharge for engineering design purposes will occur when all depression storage is filled and the soil is completely saturated. All excess precipitation will then be directed to the outlet as direct runoff. Therefore, the runoff coefficient for a submerged wetland is essentially 1.0 since evaporation from the water surface usually can be neglected during a storm event.

If the wetland catchment area or subregion is not completely submerged, then the runoff coefficient is estimated according to soil type, vegetation type, and depression capacity. Values of the runoff coefficient quoted in the literature for vegetated areas range from 0.05 to 0.35. Values of C are lower for sandy soils and higher for dense soils. The slope of the subregion also affects the value of C. Steeper slopes result in higher values of the runoff coefficient than mild slopes. Finally, the vegetation density affects the runoff coefficient. Higher plant densities reduce runoff and increase evapotranspiration, resulting in a lower runoff coefficient than for sparse vegetation.

The rational method has been used extensively in small urban watersheds where runoff coefficients for impervious materials are well known. The method is less reliable in vegetated watersheds because of the uncertainties associated with the runoff coefficient. For design of wetland hydraulic structures, the rational method is most useful for submerged wetlands with small catchment areas (less than 10 acres). The uncertainty in assigning appropriate values for the runoff coefficient increases with increasing catchment area. The assumption of an equilibrium hydrograph is reasonable for small watersheds but is rarely achieved in larger basins where the time of concentration is large.

Unit Hydrograph

In wetlands with large catchment areas, the unit hydrograph may be used to estimate peak discharge for a storm of a given duration. This method permits the engineer to design for a realistic storm of limited duration, unlike the rational method which assumes that the storm duration is equal to or greater than the time of concentration. The assumptions of the unit hydrograph method are:

a. The watershed response is linearly related to rainfall intensity

b. The rainfall during a storm is spatially and temporally uniform

c. The watershed response is independent of antecedent precipitation.

The assumption of linearity is adequate to determine design discharges for culverts and gates. These assumptions are not appropriate for design of wetland channels and habitat areas.

The unit hydrograph for a storm of a given duration can be derived by monitoring the discharge at the outlet during a single storm event. The time-discharge response recorded for a two-hour rainfall event is assumed to have a characteristic shape which is constant for all two-hour storms. The magnitude of the discharge is assumed to be linearly proportional to the rainfall intensity. Therefore, the unit hydrograph represents the basin outflow (in cubic inches per second) resulting from one inch of direct runoff generated uniformly over the drainage area at a uniform rainfall rate during a specified period of rainfall duration (Sherman 1932). This method is advantageous when little historic data are available. The response of the watershed to a design storm can be estimated from storm hydrographs measured at the outlet over a short monitoring period (approximately one year). However, the assumption of linearity is not realistic, particularly when extended to extreme flow events. The uncertainty associated with estimates derived by the unit hydrograph method is high.

Synthetic Unit Hydrographs

Before the widespread use of personal computers, a variety of techniques were developed to synthesize unit hydrographs in watersheds that have not been monitored. These methods generally rely on empirical runoff coefficients developed for a gauged watershed that are then applied to an ungauged site. These methods have proved to be of limited value because of the high level of uncertainty in these coefficients. With the advent of personal computers a large number of more accurate numerical hydraulic routing software have been developed to predict runoff generation from ungauged watersheds. Graphical user interfaces make these software packages easier to use than they once were. It is recommended that the wetland engineer pursue the more accurate and less empirical hydraulic routing techniques rather than the synthetic unit hydrograph techniques for the prediction of runoff generation when no streamflow data are available. Hydraulic routing is discussed in more detail in the following section.

Correlation Models

If sufficient data are available and the hydrology of the wetland is sufficiently simple one can sometimes correlate a continuously monitored hydrologic variable to another, unmonitored hydrologic variable. For example, streamflow at an upstream gauge may be correlated to the water level in the wetland. If so, one can construct a correlation model to predict the water level in the wetland based on data coming from the stream gauge. To construct a correlation model, an onsite monitoring program is required to measure both of the parameters of interest synoptically and over a range of hydrologic conditions. These data can then be analyzed statistically to determine if a correlation exists. Once the correlation has been quantified, measurement of the

second parameter can be discontinued. When collecting data for a correlation model it is important to include data collected during extreme events, such as floods and droughts. This procedure reduces the uncertainty of predictions made using the correlation model during extreme events. A comprehensive treatment of correlation models can be found in Kleinbaum and Kupper (1978) or in any linear regression text. An example application of a correlation model for wetland hydrology was presented by Richter (1995).

Surface Flow Hydrodynamics

Wetland engineering requires that hydraulic control structures and construction features produce a pattern of inundation that promotes the growth of beneficial wetland vegetation. The frequency, duration and depth of inundation events will have a combined effect on plant species distribution. Certain aquatic animal species exist in narrow ranges of hydraulic conditions that are related to the maximum current velocity, the minimum current velocity, the degree of vertical mixing, the salinity, and the stability of the channel substrate. The specified wetland design criteria should include the exact inundation duration and frequency requirements, the desired substrate, and the water quality (particularly salinity) specifications for the desired assemblage of vegetation and wildlife. The engineer must solicit the aid of a wetland biologist who is familiar with the desirable species in order to determine these design criteria.

Surface flow hydrodynamics and hydraulic flood routing are two techniques for analyzing the spatial and temporal distribution of surface flows in a wetland. Hydraulic flood routing is used to compute the storm runoff generated in basins or sub-basins and to route the storm runoff through a network of one-dimensional drainage channels by one of several numerical routing methods. Surface flow hydrodynamic modeling is a technique by which the shallow water equations are solved numerically in either one, two, or three dimensions. Hydraulic flood routing computes a flood hydrograph at each computational point in the system. Surface flow hydrodynamic modeling provides the depth of water and the velocity, magnitude and direction at each computational point in the system and at each computational time step. Flood routing is most commonly used to determine peak discharges, time to peak, and recession curves for complex channel systems. Hydrodynamic modeling is used to determine current patterns and flow depths in complex flow regions which may be channelized or spread over a basin with an irregular topography.

Methods of Analysis for Surface Flow Hydrodynamics

Hydraulic Flood Routing

Nonlinear methods of hydraulic routing within the wetland watershed are useful for determining peak discharge, for predicting the temporal response of the wetland to a flood event, and for analyzing spatially and temporally heterogeneous hydrologic events. Hydraulic routing utilizes physically based equations of open channel flow to account for channel resistance and storage capacity. The degree of accuracy for these methods is high provided that the appropriate forms of the equations are applied, and the hydrogeomorphic data supplied to the model are accurate. The data required for hydraulic routing include:

a. Temporal and spatial distribution of rainfall for the design storm

b. Spatially distributed runoff coefficients, or inflow storm hydrographs for each of the catchment subregions

c. Channel and floodplain cross sections over the basin

d. Channel roughness characterization

e. Downstream boundary condition

Data required for hydraulic routing must be generated as part of the wetland design project. Recent developments in computer software have made processing and analyzing geomorphic data much easier than in the past. Geographical information systems (GIS) and graphical data displays provide special purpose tools that are ideally suited to watershed analysis. A software package called GEOSHED has been developed to analyze hydrologic characteristics of catchment basins (Richards 1993b).

The effort required to perform hydraulic routing can result in a net cost savings over designs based on the rational method or the unit hydrograph, in spite of the greater amount of data required. A large degree of uncertainty is associated with both the rational method and the unit hydrograph method. This uncertainty can result in overestimation of the design discharge and the need to specify a high factor of safety. A project that is overdesigned for the site will have concomitant excess material and construction costs. Design phase site evaluation and data collection is a cost-effective approach because time charged in preconstruction analysis is typically cheaper than time charged during construction.

Hydraulic routing models are sufficiently complex that a computer program is employed to perform the calculations. Computer models also permit the engineer to specify more spatial heterogeneity in the catchment subregions than can be considered using hand calculations. Thus, the physical characteristics of the wetland watershed can be accurately represented, which reduces uncertainty in the calculated peak flows. Hydraulic routing models represent drainage of the catchment area as flow through a network of one-dimensional conveyance channels.

The simplest routing methods, such as the Muskingum Method, are based on continuity of mass. Travel time through a reach is estimated from the channel storage capacity. In practice, the travel time coefficients for Muskingum routing are back-calculated from observed storm hydrographs. Models based on physical characteristics of the watershed are preferred for wetland restoration projects because measured hydrographs are rarely available. Hydraulic routing can be calculated by simultaneous solution of the equations of momentum conservation and mass conservation. Examples of physically based routing models are the diffusive wave model, the kinematic wave model, and the dynamic wave model (Henderson 1966). Many public domain computer programs exist to facilitate hydraulic routing calculations, including the model HEC-1 (Hydrologic Engineering Center 1981). The GEOSHED software package integrates a GIS database with HEC-1 to perform hydraulic routing calculations in spatially heterogeneous watersheds (Jones et al. 1990; Nelson et al., 1993; Richards 1993b).

A friction coefficient must be specified for solution of the momentum equation in open channel flow. This coefficient describes the resistance force caused by the bed and the vegetation. Manning and Chezy coefficients are widely used. In wetland watersheds, resistance due to vegetation is high, and the effect of the vegetation on the flow can extend throughout the water column. Manning's n values from 0.1 to 0.35 have been reported for hydraulic routing in wetlands, with higher values corresponding to greater vegetation densities. Recent research indicates that Manning's n values in vegetated channels are not constant, but vary with flow velocity and depth. (Roig 1994).

The methods discussed herein for computing the peak discharge in an ungauged watershed include the rational method, the unit hydrograph, and hydraulic routing. Of these methods, hydraulic routing provides the highest degree of accuracy and reliability. The rational method is adequate for small wetlands that are largely inundated during the design storm, and where overdesign of the structure will not result in excessive construction costs. Collection of geomorphic data for hydraulic routing can be a worthwhile investment because peak flows are more accurately estimated. A more efficient design for the control structure will result.

TABS-MD and FastTABS

A two-dimensional, vertically averaged hydrodynamic model can be used to predict circulation and ensure that hydrodynamic conditions meet biological requirements. Parameters such as channel dimensions, gate operation schedules, and weir heights can be adjusted within the model to test the design against biological design criteria (Richards 1993a). Example applications of a two-dimensional hydrodynamic model are presented by Evans and Roig (1995).

The Waterways Experiment Station has developed a multidimensional, vertically integrated finite element model system (TABS-MD) which will accurately model the surface flow in wetlands. In addition, a graphical interface (FastTABS) has been developed which permits the user to rapidly analyze various wetland plans. FastTABS provides both pre- and post-processing functions. The flexibility to modify the numerical grid resolutions is particularly useful in wetland environments where engineers must address a variety of environmental and operational questions.

The TABS-MD hydrodynamic modeling system consists of a group of models which include hydrodynamics, transport, and sedimentation. The two-dimensional, vertically averaged hydrodynamic model is based on RMA2V, a model originally developed by Norton, King, and Orlob (1973). It has been further developed by the WES Hydraulics Laboratory and is referred to as RMA2-WES. RMA2-WES computes the water surface elevations and vertically averaged flow velocities using the finite element method. Both steady-state and dynamic solutions can be obtained. Boundary conditions are specified as flow rates, water elevation, or water velocity. A fully implicit finite difference discretization in time permits the modeler to choose a variable time step that can accurately capture the changes in the boundary conditions over time. Hydrodynamic parameters such as Mannings n and eddy viscosity can be defined by element type or by individual elements or nodes. These parameters may be temporally and spatially varied. An essential modeling feature for wetland modeling is the ability to simulate intermittent flooding and draining of the marsh surface over the simulation period.

The FastTABS computer program was developed by the Brigham Young University Engineering Computer Graphics Laboratory in cooperation with the Hydraulics Laboratory, Waterways Experiment Station. It is a graphic pre- and post-processor for the TABS-MD hydrodynamic modeling system. This allows the wetland designer to quickly and easily construct a finite element mesh, generate hydrodynamic results with RMA2-WES, analyze the results for validity, and perform modifications. Results can be presented on an areal basis by contouring (either by contour lines or color shading) or on a nodal basis via time-series plots of water surface elevations and velocities. Results can be saved as graphics files and in spreadsheet format.

Wetlands Dynamic Water Budget Model

A link-node model for wetland hydrology and hydraulics has been developed by Walton et al. (1995). This model incorporates interactions between surface water flows, vertical processes, and horizontal groundwater flow. Some of the processes that can be described with this model include channel and overbank flows, tidal forcing, riverine inflows, upstream basin flows, wind shear, flooding and drying, bottom friction, hydraulic structures, canopy interception and drainage, infiltration, surface water evaporation, soil water evaporation, transpiration, and variably saturated horizontal groundwater flow. The model has been used to evaluate alternative wetland management scenarios.

Other Hydrodynamic Models

The modeling systems described above are not the only hydrodynamic models available for shallow water flows. However, few other commercially available models are targeted for wetland applications. Other hydrodynamic models for wetlands are being developed in the academic and research arena, some results of which are published in current scientific journals. These models are generally available by contacting the author of the model.

Groundwater Interactions, Seepage, and Infiltration Analysis Methods

Naturally occurring wetlands frequently exist in areas with a high water table. Some wetlands occur because the water table actually rises above the land surface, contributing to the surface flow. Other wetlands occur in areas of active groundwater recharge because the abundant and widespread surface flow in the wetland saturates the soil and eventually intersects the water table. For the purpose of wetland design the engineer would like to know whether the net seepage is positive (from the surface water to the groundwater) or negative (from the groundwater to the surface water). This piece of information is frequently obtained by means of a water balance as discussed above. A water balance accounts for all measurable sources, sinks, and storage of water in the wetland. Two hydrologic components, seepage and evapotranspiration, are typically not measured. If the engineer can estimate evapotranspiration from a knowledge of the vegetation cover, then the only unknown component of the water balance is seepage, which can be found by difference. The other methods that are available for estimating groundwater contributions are much more complex. Groundwater flow modeling may need to be used when groundwater is determined to be the major source of water to the wetland, when there

is insufficient water supply data to prepare a proper water balance, or when the site is large and spatially complex. Monitoring wells can be helpful for determining the piezometric surface which sometimes gives an indication of the flow direction.

Subsurface Data

Networks of monitoring wells are maintained by certain local, state and federal agencies for special purposes. Irrigation districts often monitor aquifer levels to determine irrigation water supplies. Municipalities that rely on groundwater for domestic and industrial water supply also monitor the local aquifer levels. The USGS and USBR maintain monitoring networks in some regions for research purposes and for environmental impact assessment. Unfortunately, there is no central clearinghouse for groundwater data that encompasses all participating agencies. State departments of water resources are a good source of information about the extent and availability of existing groundwater data.

Modeling

Water Balance Models

A water balance is a systematic method for presenting information about sources, sinks, and storage of water within a bounded region. Many wetland design problems are concerned with the control and management of surface water resources. But the engineer must know the magnitude and direction of seepage in order to design appropriate engineering works. The water balance is a suitable method for determining seepage if sufficient surface water data are available and if the flow behavior of the aquifer will not be significantly impacted by the engineering design. A water balance requires data about the annual precipitation, streamflows, and evapotranspiration. If the watershed is ungauged, a surface water monitoring program must be initiated in order to construct a water balance. The Hydrologic Engineering Center (1980) has published a guide for the preparation of water balances.

Infiltration Models

Infiltration of surface water into the soil is important for determining rainfall-runoff relationships. It is also important for computing the groundwater storage volume and soil moisture. Most rainfall-runoff models compute the infiltration as a percentage of the total precipitation reaching the soil surface. This percentage varies depending upon the soil type, the vegetative cover, the land surface slope, the antecedent soil moisture conditions, and the depth to groundwater. A variety of methods have been developed to determine infiltration rates based on these factors. An overview of the most frequently used models is provided in Maidment (1993). These models include the SCS Runoff Curve Number model, the Horton model, the Holtan model, and the Green-Ampt model.

Groundwater Models

Groundwater modeling is a technique by which the equations governing groundwater flow are solved in one, two, or three dimensions over a spatially discretized domain. A computer program is employed to solve the resulting system of equations. Groundwater modeling will not

be necessary for most wetland restoration and creation projects. The exceptions are wetlands where groundwater is the principal source of water and the flow of water in the aquifer is likely to be impacted by the wetland engineering works. A complete discussion of groundwater modeling is beyond the scope of this handbook. An introduction to the subject has been published (Bear et al. 1992).

5-3 Design of Water Control Structures[1]

Introduction

Man-made wetlands require some means to control the quantity and depth of water at a given location. Consequently, hydraulic structures are a basic part of creating, restoring, and enhancing wetlands. Throughout history, man has been building structures to contain or control water. The result is that methods have been established to achieve this purpose and the "hard" engineering is relatively straightforward once the conceptual design has been established. Wetland structures generally fit into one of four categories: water containment structures, water control structures, erosion control structures, and habitat structures. This chapter describes water containment structures and water control structures; Chapter 5-4 discusses erosion control structures.

Inflow Design Considerations

Water is the basic component of a wetland site. The source of the water may be natural runoff delivered to the site by stream or overland flow, groundwater, or through tidal action. In such cases, the quantity and timing of the inflow is uncontrolled but may be statistically estimated through appropriate hydrologic analysis. Water may be delivered to the site through rigid controls such as by pumping or through an operated control structure (Watson and Hobson 1988). The inflow to the site may be semicontrolled through the use of restricted entrances such as small or gated culverts or by raised weirs that allow inflow to the site only if the water supply is above some fixed or adjustable elevation. Tidal inflows may be moderated through the use of limited width openings or weirs in levee systems (Chabreck 1976). If structural options are used for the inflow, the design should accommodate changing conditions on the supply side of the structure as well as on the wetland side. Channels or other distribution systems may be required on the wetland interior to help spread the flow throughout the site.

[1] By Charles H. Tate, Jr. and Trudy J. Olin

Structure Types

Water Containment Structures

Containment structures confine water, sediments and in some cases nutrients and contaminants within a wetland. Permeable containment structures capture sediments, while allowing water to pass through the structure. Impermeable containment structures serve to impound water within the wetland and inflows must be discharged through pipes or culverts or over weirs or spillways. Common containment structures include:

a. levees

b. dikes

c. embankments

d. dams

e. gabions

f. fabric bags

g. walls

h. liners

Levees, dikes, and embankments are all terms used to describe an earthen structure used to contain or control the direction of water flow; Section 4 describes the characteristics of these earthen structures. As in Section 4, the function of these structure is similar for the applications described here. Thus, “dike” in the following paragraphs refers to all of these structures.

Dikes can be effectively used in a wetland system to control flow paths and to minimize short-circuiting. Often these structures are used to subdivide a wetland area to allow operational management, to control flow paths (Watson and Hobson 1989), or to establish differing habitat areas. These are the structures normally used in wetland construction due to cost, material availability, and material suitability. General engineering practice is to build dikes with a narrow base, steep sides, and a uniform cross section limited only by stability requirements. These guidelines may be inappropriate for wetland construction. The side slopes that are under the water should be varied to create variable widths of emergent vegetation (Grimball 1992). The slopes above the water surface should be designed based on the wetland functions such as habitat. Slopes that are too steep or too flat may not be suitable for specific habitat. Variable slopes would accommodate multiple habitat requirements. The top width of dikes should include maintenance needs such as equipment access and patrols. Usual causes of dike failure are overtopping, undermining, sloughing, piping, or seepage along water control structures placed through the dike. The design of the dike should eliminate these dangers as much as possible (Soil Conservation Service 1992). Erosion control should be provided where the dike is subject to a high energy environment due to waves or high velocity flow. This protection may be supplied by

vegetation, riprap, or geotextiles or a combination of these. Burrowing animals may undermine dikes and should be accounted for in the design and maintenance process. In addition to containing the water, dikes may be used to direct or control the flow path through the wetland site. Such dikes may extend from the boundary or containing dikes or may be separated from the boundary to provide isolated habitat. Clusters of islands may also be used.

Water containment structures may also be constructed with walls made from wood, steel, concrete, fabric bags, gabions, or any other suitable material. Liners may be used with pervious construction materials to form an impermeable surface. Table 5-4 lists some structure types with materials and design considerations.

Water Control Structures

Some type of water level or flow control structure is usually required to control the hydraulic regime of a restored or created wetland. Water control structures control the surface elevation, volume, direction, depth and velocity of flow into and out of a wetland. Inlet and outlet control allows flexible operation of the wetland through volume and flow rate control in conjunction with the duration and season the wetland will be inundated. Such flexibility may be essential for establishment and survival of wetland vegetation and to attract the desired wildlife. Natural open channels are the simplest water inlets and outlets. As necessary, these channels can be protected using concrete or stone lining, and flow control can be added with a control structure such as a weir.

Most wetland designs will incorporate some type of inlet or outlet structure, flow splitter, or diversion structure for controlling the flow of water through the wetland. These structures can be completely passive, or require manual or automated operation. Regardless of their construction or operation, all water control structures require periodic maintenance, occasional clearing of obstructions, and performance monitoring.

Outflow is usually controlled by overflow structures, submerged culverts, or, on rare occasions, by pumps. Overflow structures usually consist of overflow weirs constructed using flashboards, sills, spillways, and drop inlet pipes. Culverts often have some form of adjustable flow control device on one end. These structures may be located in the ponded area or remotely located in the embankments of the wetland. Controlling the outflow from a wetland is directly dependent upon the wetland function being obtained and the ultimate use and design of the wetland system. The wetland area may be designed for flood conveyance which would require sufficient capacity to temporarily store runoff for later release. The pool elevation might need to be regulated to maintain proper water depth for specific habitat needs. The possibility of total drawdown of the wetland might be necessary for wetland management purposes. Table 5-5 presents common water control structures and important design considerations for those structures.

For impoundments with dikes 1 ft or less in settled heights the Soil Conservation Service (SCS) states that vegetated spillways may be used in lieu of structures with dewatering done by cutting the dikes. For impoundments with dikes more than 1 ft in settled height, the SCS recommends the following outflow structures:

Table 5-4
Water Containment Structures and Design Considerations

Structure Type	Materials	Design Considerations
Dikes	Earth - silty or clayey sand, sand or gravel: permeable dikes will be constructed from coarsest material, impermeable from finer materials	• Dike height limited by foundation stability and by base width required by angle of repose of dike materials • Erosion susceptibility of materials should be considered - slope erosion and toe scour may occur - dike protection may be required under certain conditions • Soil characteristics as related to dike construction are listed in SCS (1992) • Low cost structure Reference: Eckert, Giles, and Smith 1978, SCS 1992, US Navy 1971, Hammer and Blackburn 1977
Cofferdam	Cells filled with granular material. Cells may be constructed from any material, but usually steel sheet-piling.	• Cofferdams are used as temporary structures in construction where isolation of a site from the adjacent water body is required. Fill can be placed behind permanent cofferdams. • Maximum height 60 feet under optimum conditions (maximum economic height typically about 20 feet) (Eckert, Giles and Smith 1978) • Good erosion resistance • Susceptible to toe scour • High cost structure Reference: Eckert, Giles, and Smith 1978, US Navy 1971
Gabions	Galvanized or coated wire baskets filled with 4- to 10-inch-diameter rock and stacked and wired together	• Gabions form a somewhat permeable wall: may not retain fine materials unless paired with filter fabric • Maximum height approximately 10 feet with multiple thicknesses or counterforting (Eckert, Giles and Smith 1978) • Susceptible to toe scour when placed on erodible substrate: flexible gabion apron may be required. • Moderate cost structure Reference: Eckert, Giles, and Smith 1978
Geotextile Containers	Synthetic fabric bags filled with sand, sand-cement, concrete or fine soils or sediments.	• Geotextile containers can be used for dike construction, island construction and for training structures (see also erosion control structures) • Maximum height typically 16 feet or less for freestanding structures (Eckert, Giles and Smith 1978) • Permeable fabric bags must be paired with filter cloth or a stone filter layer to retain fine materials • Propagation of natural vegetation through containers filled with fine materials can occur • Good erosion resistance; Articulated mattress at toe may be required to prevent toe scour • Low cost - 3-5 year life for sand filled, sand-cement bags have extended life, and concrete filled bags can be considered permanent (Eckert, Giles and Smith 1978) Reference: Eckert, Giles, and Smith 1978, Fowler and Sprague 1993, USACE 1977

Table 5-4 (continued)
Water Containment Structures and Design Considerations

Structure Type	Materials	Design Considerations
Cantilevered Retaining Wall	Arch web or Z-type steel sheet piles; wood; concrete	• Maximum wall height approximately 15 feet under optimum embedment conditions; adequate embedment in firm bottom strata required (Eckert, Giles and Smith 1978) • Good erosion resistance; Toe scour occurs under some conditions • Moderate to low cost structure/sheet piles can be rented for temporary structures Reference: Eckert, Giles, and Smith 1978, US Navy 1971
Anchored Retaining Wall	Arch web or Z-type steel sheet piles; wood; concrete	• Maximum wall height approximately 40 feet under optimum embedment conditions; adequate embedment in firm bottom strata required (Eckert, Giles and Smith 1978) • Good erosion resistance; Toe scour occurs under some conditions • Moderate to high cost Reference: Eckert, Giles, and Smith 1978, US Navy 1971
Liners	Clay or synthetic materials	• Transport of leachate or very fine particles and associated contaminants into groundwater can be restricted with the use of appropriate lining material Reference: Koerner 1998

a. A straight drop structure, which may be equipped with removable stoplogs, constructed of treated timber, metal, sheet piling, rock, or concrete.

b. A pipe provided with a swivel elbow and riser.

c. A pipe drop inlet structure, which may be equipped with a gate, valve, or plug for controlling flow.

d. A pipe provided with a perforated riser.

Hydrologically isolated wetlands that do not have a contributing drainage area will require outlet control structures only if groundwater inflow and precipitation will likely exceed evapotranspiration and the available water storage volume.

Sediment

Reduced flow velocities found in wetlands, whether natural or man-made, allow sediments transported into the wetland with the inflow to settle to the bottom. Trapped sediments occupy part of the wetland volume, thereby reducing its effectiveness at removing future sediments from the inflow. Because nutrients are often associated with the sediment particles, incoming sediments can increase the vitality of the wetland. However, wetlands which receive large

Table 5-5
Water Control Structures and Design Considerations

Structure Type	Function	Materials	Design Considerations
Distribution Header	Distribute flow over width of wetland	Perforated pipe Pipe w/fixed or movable tees Channels with multiple outlets	• Uniform flow distribution is important to subsurface flow systems (Watson and Hobson 1989) • A distribution header may be located above or below ground. Above ground headers can feed into a gravel trench for uniform distribution. Reference: Watson and Hobson 1989
Headgates	Water level regulation	Sheet metal	Headgates are typically a simple device consisting of an angle iron frame fitted to the face of the inlet flume or pipe, and extending above it. Within the frame is mounted a sheet metal "gate" which is raised and lowered by means of a threaded rod and flywheel.
Pipes/ Culverts	Water level regulation	PVC Galvanized Concrete Cast Iron	• Relatively maintenance free • Suitable for water level regulation for small impoundments • Submerged inlet - perforated collector pipe configuration or short riser with hooded inlet - may have swiveling elbow to horizontal outlet pipe • Swiveling pipe structure limited to small diameter pipe and small flows • Inlet may be susceptible to sedimentation Reference: Hammer 1992
Swales with and without Swale Blocks	Vegetated open channel for transport and infiltration of storm water	Earth, natural vegetation	• Swale blocks are simple earthen berms placed perpendicular to the flowline of the swale. Swale blocks are used to create temporary detention within a section of swale to permit time for infiltration. • Inlet/oulet structures sized to pass maximum-design storm flows, sited to minimize short circuiting Reference: Wanielista et al. 1986
Parshall Flume	Utilized to measure and control flow rate	Concrete Sheet metal Treated or rot resistant wood	
Flashboard Culvert	Utilized to control water level and outflow	Metal pipe riser with flashboard (stoplog) fittings/wood stoplogs	• Asphalt or other coatings necessary in corrosive waters • Typically risers no larger than 0.8 meter (Hammer 1992) • Not suited to large flows or widely varying flows • More susceptible to blockage than stoplog structure Reference: Hammer 1992

Table 5-5 (continued)
Water Control Structures and Design Considerations

Structure Type	Function	Materials	Design Considerations
Weir	Utilized to stabilize water levels or prevent complete drainage of wetland with tidal fluctuation. Aquatic plants proliferate, waterfowl habitat is increased, navigable waterways are maintained. (Nyman, Chabreck and Kinler 1993)	Sheet piling Wood Stone Concrete Vegetated earth	• Effects: Species diversity and long-term vegetal mortality effects due to limited drainage should be considered. Reference: Nyman, et al. 1993
Stoplog Structures	Utilized to maintain variable water levels in impoundment	PVC Wood Concrete walls/ wood stoplogs	• Water level control achieved by adding or removing stoplogs • More expensive than flashboard culverts or swiveling pipe structures - less susceptible to blockage, better handles large flows - utilized in larger impoundments (Hammer 1992) Reference: Watkins 1992, Hammer 1992
Flow Splitter	Direct or distribute flow within a single wetland "cell", or between multiple cells	Pipes, Flumes Wiers, Slotted baffle plates, Perforated distributor pipe	• Typically, orifices are parallel and of equal size, at the same elevation • Flumes minimize clogging problems for high solids applications Reference: Watson and Hobson 1989
Baffles/ Finger Dikes	Direct flow, increase flow path to maximize retention time, optimize length to width ratios, minimize short-circuiting	Earthen dikes Steel plates Riprap Gabions	Baffles increase the retention time by reducing short-circuiting of flows and are useful for maximizing sedimentation efficiency and other processes, such as biodegradation, that are a function of retention time

sediment loads can fill rapidly with sediment. Accordingly, a provision to either remove the sediments prior to reaching the wetland or an occasional dredging of the wetland may be necessary. Most sediments can be removed by employing a settling basin upstream of the wetland to protect it from the incoming sediment load. The settling basin should not be considered as habitat area and plans must be made to remove the sediments from the settling basin periodically. Easements and methods of access for sediment removal should be included in the design along with the information that this portion of the site will be subjected to construction activity on some periodic basis.

Fine sediments take longer to settle and will often pass through a settling basin into the wetland site. Over time there will probably be a loss in water volume as these sediments fill the site. Long period renovation may need to be considered in the design based on the fine sediment load to the site. Vegetation reduces the flow velocity due to the resistance of the roots, stems, and leaves to the flow and fine sediments freqently deposit in vegetated areas of the wetland.

In some watersheds, one or several contaminants may be associated with sediments coming into the wetland. Since most contaminants are tightly bound to fine sediment particles they will be found in areas of sediment deposition. If contaminants are likely to exist in significant concentrations, their impact upon the ecosystem and the likelihood of future remediation requirements should be carefully considered in wetland planning and design.

Outflow

Outflow structures may be of prime importance especially where there is little or no inflow control. Outflow structures are of two basic types. One is where water flows over a control that is at a fixed or variable elevation. Drop pipes, flashboards, and spillways are examples of this type of structure. The other is where flow passes through a bottom or midlevel outlet such as a culvert or permeable material such as riprap (Swanson, Franzen, and Manning 1987). The "hard" engineering design and structure (or structure combination) selection should be based on the site requirements for water level fluctuation and flow. If the requirements are for minimum water level fluctuations but with large variations in flow then an overflow structure with a long crest length may be required. If, instead, floodwaters are to be stored, then large fluctuations in the pool depth must be included in the design of the outlet structure(s) and the dike system. A design event should be selected around which the outlet structure(s) is designed. A failure point and mechanism should be designed into the system to minimize damage for events that exceed the design event. The failure point should be located where it can easily be repaired. For structures where the water flows over a control, the design concerns will be how long to make the control, how high will the water get over the control, and what value to use for the discharge coefficient for the control. The materials used for the control are typically wooden boards, metal culvert pipe, plastic pipe, riprap, concrete, or reinforced vegetation. All of these types of materials and structures are subject to having the discharge coefficient vary due to debris accumulation, changing vegetation densities, or material degradation. Accordingly, the design process should look at both the new and the aged conditions. Bottom or midlevel releases are typically accomplished with culverts that may be gated to control the outflow or by gated structures such as sluice gates in a dam. Uncontrolled culverts may be designed such that the higher flows are controlled by either the entrance or the outlet of the culvert. Again, debris or other blockages may affect the actual flow through such structures. There have been hybrid structures where a flashboard located in the center of a dike was used to control the water level with the water getting to the flashboard through a culvert.

Operations and maintenance must be included in the design process. There is little rationale for designing an outlet structure that requires frequent operations if the personnel and financial resources are not available. Wetland sites should be designed to be relatively operation- and maintenance-free. Materials used for the outlet structure should match the site conditions. Acidic water may cause a corrosion problem for metal or concrete structures. The addition of fly ash in

the concrete mixture may lengthen the life of concrete structures in such an environment. Pipe abrasion by coarse sediments, particularly for plastics (National Academy of Sciences 1978), can be a problem and should be considered in material selection. If the outlet structure is designed and constructed for a specific design life, the design should include plans to replace the structure as needed.

Life safety should always be considered when selecting an outlet structure. Many wetland areas will be used for authorized or unauthorized recreation. Attempts to exclude unauthorized access to wetland sites are usually doomed and should not be the primary safety feature of a design. Strong roller action at outlets and unobstructed culvert openings should be avoided.

Basic Design Methods

Basic design of outlet structures includes selecting overflow or subsurface structures, selecting a design event for hydraulic design, and selecting appropriate materials for the outlet structure.

Overflow Structure Design

The portion of an overflow outlet structure over which the water passes is referred to as a weir. For most circumstances, as long as the water downstream of the weir is below the crest of the weir then free overflow will occur at the weir. For free overflow conditions the following equation can be used to design a weir for specific conditions.

$$Q = CLh^{1.5} \tag{5-2}$$

where:

Q = flow rate, m^3/sec
C = discharge coefficient, $m^{0.5}$/sec
L = the effective horizontal length of the weir in feet, m
h = the height of the energy line above the weir in feet, m

The water velocity in wetlands, as in most impoundments, will be sufficiently slow so the water surface can be used as an approximation of the energy line approaching the weir (h). The water surface elevation (about five times the depth of flow over the weir) should be used as the elevation of hydraulic grade line (HGL) at the weir because of potential flow acceleration effects at the weir. If the approach velocity to the weir is greater than 0.5 m/sec, the velocity head should be added to the water surface to determine the elevation of the energy grade line (EGL) at the weir. The velocity head is defined as

$$velocity\ head = \frac{V^2}{2g} \tag{5-3}$$

where:

V = horizontal velocity of flow, m/sec
g = gravitational acceleration, m/sec^2

The effective length of a straight or semistraight weir is the actual length minus the effects of end contractions and the width of any piers or other obstructions along the length of the weir and associated end contractions. The weir length should be modified as follows to determine the effective weir length (Chow 1959):

$$W_L' = W_L - \Delta W_L \tag{5-4}$$

where:

W_L = actual weir length, m
W_L' = actual weir length, m
ΔW_L = change in weir length (note that all changes *shorten* the weir length), m, calculated as:

$\Delta W_L = 0.05h$	for each end contraction
$\Delta W_L = 0.1h + W_{obstruction}$	for blunt obstructions
$\Delta W_L = 0.05h + W_{obstruction}$	for streamlined obstructions

and

$W_{obstruction}$ = width of weir obstruction at widest point, m.

The discharge coefficient (C) depends on the efficiency of the weir in question and varies the water depth flowing over the weir and the water depth just in front of the weir. For sharp-crested or nearly sharp-crested weirs with free overflow, the value of C is approximately 1.83 m$^{0.5}$/sec (Streeter 1971). For smooth broad-crested weirs such as wide spillways, C is approximately 1.7 (Streeter 1971).

If the weir is submerged from the downstream side, it acts as an orifice and the discharge coefficient C changes based on the amount of submergence. Complex design charts for orifice outlets can be found in Chow (1959). Submerged conditions should be avoided if the upstream water level is to be controlled by the weir.

Drop inlets such as riser pipes can be treated as weirs with the circumference of the inlet being the weir length. This concept works up to the point that the overflowing water creates a submergence effect. This effect generally starts when the water depth above the weir crest equals 45 percent of the radius of the riser pipe (USACE 1989).

Submerged Flow Structure Design

Submerged outlet structures have several methods for design depending on the type and complexity of the outlet structure. Uncontrolled culverts are one of the more popular outlet structures used. The Federal Highway Administration has developed design methods for most culvert conditions based on extensive laboratory and field investigations (USFHWA 1984). Gates or other controls may be added to culverts to allow variable control of subsurface releases. Restrictions, if used, should be placed on the upstream end of culverts to prevent clogging of the culvert by sediment or debris. Design information for large outlet structure gates may be found in EM 1110-2-1602 (USACE 1963) or hydraulic engineering handbooks such as Chow (1959). For smaller controls such as gate valves trial setting may be required to achieve the desired flow control although King and Brater (1963) has information that may be useful. Porous structures may be used that pass flow from all or a fixed portion of the water column. Rock is probably the most common material for this type of structure but any porous medium or construction would fall into this category. Flow conveyance is difficult to determine for these structures due to unknowns in pore sizes and flow paths. The potential for clogging should be investigated prior to using this type of structure. Where water flows into and out of a wetland through the same structure, porous structures can be used to attenuate the water surface fluctuations. Groundwater flow analysis methods would be helpful in designing such structures.

Design Event

Hydraulic structures are designed to routinely pass flow up to some design event that is usually based on a statistical probability of occurrence. The statistical event varies based on the type of structure being built, the damage costs that would result due to structural failure, and the risk to life due to failure. Many Soil Conservation Service projects are based on an event that has a statistical probability of occurring once every two years. Flows that exceed this event may cause flooding. Most US Army Corps of Engineers projects are designed for probabilities of once every 100 years or greater. This implies that the design conditions will be exceeded less often. Design event determination for a wetland site should be based on cost effectiveness and the cost of failure. Once the design event has been determined and the hydraulic structures designed, a failure point and mechanism should be determined and included in the project. Failure points should be located where reconstruction can easily occur, keeping in mind the available access for equipment and the cost of reconstruction. The mechanism could be a lower reach of a dike or an overflow spillway that would wash out without endangering the remainder of the dike system or other structures. An option to a failure point could be to construct overflow structures to allow greater than design flows to pass through the project without causing failure of the structures. The overflow structures should be designed to handle flows up to some statistically probable event, keeping cost effectiveness in mind.

Materials

Due to cost considerations, most wetland outlet structures will be constructed with standard materials such as steel, cast iron, concrete, plastics, and wood. In determining which materials to use considerations should include the desired life span of the project, the potential for mechanical degradation, and the potential for chemical degradation. These considerations may

also dictate the type of outlet structure. Steel and concrete are subject to abrasion and acid attack. Plastics and aluminum may be more resistant to acid attack but are less resistant to abrasion. Asphalt or other coatings will usually increase the life of steel products. Stainless steel is more resistant to attack but is expensive. Wood may last depending on the type of wood and its exposure to wet/dry cycles. Natural materials such as rock can be degraded through freeze/thaw cycles. Life cycle costs should be evaluated when selecting construction materials realizing the potential replacement costs and cycles for replacement of outlet structures.

Structure Selection

Existing hydraulic structures are extremely varied due primarily to the need to satisfy highly varied design constraints. The final choice for the design is limited only by the imagination of the designer using the constraints mentioned above. However, several basic structures are normally used.

An adjustable weir constructed with removable flashboards is very common. Historically these structures have been constructed with metal or wood frames and wood flashboards. Recently all-plastic structures have been used (Watkins 1992). These structures are usually sized such that one person can handle the flashboards to make adjustments to the water level in the wetland site. If longer weir lengths are needed then several individual weirs can be included in the design. Spillways or overflow sections of dikes are often used with the spillway surface constructed of an erosion-resistant material such as concrete or rock. Vegetation can be used for spillway surfaces based on the amount and duration of flow. Drop inlets constructed with corrugated metal pipe are often used for ponded wetlands. Trash or debris protection should be included in the design of these structures to prevent clogging. Occasionally horizontal pipes will be placed at the desired water surface elevation to pass flow directly through a dike.

Simple corrugated metal. concrete, or plastic pipe culverts are the most common subsurface outlet structures. Large wetlands that may pass large flows sometimes have large outlet structures similar to those found at dams. Porous structures are not common but may be useful in tidal areas or along streams subject to frequent flooding.

5-4 Shoreline Protection and Erosion Control[1]

Introduction

Shorelines and structures associated with created and restored wetlands are often exposed to the erosive forces of nature. Wetlands invite the use of bioengineering for erosion protection, and there is a greater emphasis on project aesthetics than in other types of erosion control projects. Bioengineering alone cannot provide adequate protection for some wetlands and must be combined with other forms of erosion protection. A design goal for a wetland protection project should be to use the minimum amount of structural protection necessary. Innovation is often the key to an appealing and successful project.

This chapter presents considerations for project planning and lists protection alternatives that might be considered for shoreline protection and erosion control. The text provides a checklist for things that need to be determined when planning wetland protection. The chapter does not replace more detailed erosion protection engineering manuals, many of which are referenced here. Neither does it replace the use of qualified consultants, especially ones familiar with the type of erosion problem of concern, or the particular area where the problem is occurring. Many factors such as public safety, economics, aesthetics, demographics, governing agencies, climate, strength of the erosion forces, and geology play an important role in determining the final protection alternative selected and designed. The blending of these factors requires judgment which can only be obtained from training and experience.

A good understanding of what needs to be considered for an erosion protection project, reflection on what successful techniques have been used in the past, and the experiences that can be borrowed from others are much more useful, beneficial, and efficient than any handbook.

Steps to Develop Protection for Wetland Erosion

The development of shoreline protection and erosion control measures requires several steps. These steps are:

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[1] By Jack E. Davis, Steven T. Maynord, John McCormick, and Trudy J. Olin

a. Understand the system and determine the mechanisms of erosion

b. Consider general design requirements

c. Develop a list of alternatives to protect against the cause of the erosion problem

d. Design the protection

e. Estimate the costs of the project

f. Construct, inspect, monitor, and maintain the project

In many cases there is significant overlap or iteration between these steps. The steps are discussed further below.

Understanding the System and Determining the Mechanisms of Erosion

Determining the mechanism that is leading to erosion of a wetland is the most important step in the design process and may also be the most difficult or uncertain. This is true particularly at sites where wetlands are being created and no historical perspective is available. An expert is often needed to predict or identify erosion mechanisms.

In selecting sites for creating riverine wetlands or in protection of existing riverine wetlands, one must be cautious about locations that are experiencing instability over long reaches of the system. For example, in streams where equilibrium has been disrupted by changes in the basin such as altered water levels at the downstream end, altered flowrate, or altered sediment inflow. System instability is often indicated when the channel bed elevation is degrading which can be observed by the presence of scarps or headcuts moving upstream through the system. Similar system instabilities may be associated with tidal wetlands where instabilities in the wetland are part of a much larger regional instability. When system instability is present, a comprehensive treatment plan is required and expert consultants should be retained. Solutions to system instabilities are outside the scope of this handbook.

Local instability is erosion that is occurring in a system that is otherwise in equilibrium. Local instability includes bank erosion that is part of the natural erosion process (caused by stream migration or waves) occurring at isolated locations such as open-water headlands, river bends and constrictions, and reaches around structures. This handbook addresses local instability.

Causes Versus Mechanisms

A differentiation between erosion causes and mechanisms must be made. Causes are the action or events that create forces on the wetland experiencing erosion. Mechanisms are the processes through which the bank fails. For example, a recreational boat causes waves that strike

the bank and lead to failure through the mechanisms of tractive force and turbulence removal. The cause of erosion was boat wakes while the mechanism for erosion was tractive forces and turbulent removal of sediments. Hydropower operations cause water level changes that can lead to failure through the mechanism of piping. The distinction between causes and mechanisms for erosion is important because protection methods are generally designed to address mechanisms and not causes. The following presents a list of causes of local instability that occur in tidal and riverine environments.

a. **Wind Waves** are often the dominant cause of local instability in tidal wetlands and along fringe wetlands in lakes. The zone of attack from wind waves is near the waterline but fluctuating water levels expand the zone of attack over larger portions of the bank. Waves can undercut banks leading to mass failure. The failed material piled at the toe of bank is washed away by the waves and the undercutting cycle starts over.

b. **Boat Waves**, like wind waves, can cause instability along wetland shorelines. Boat waves differ from wind waves in their size, frequency and duration but are otherwise similar with respect to their zone of attack and erosion mechanisms.

c. **Boat-Induced Currents** can cause instability in wetlands especially where large commercial vessels travel in narrow, confined waterways or where large, commercial vessels travel near banklines in larger waterways. Boat-induced currents occur from the propeller jet and from the displacement effects of large vessels traveling in relatively confined waterways. The zone of attack can be both the bottom and sides of waterways. The erosion mechanism is the tractive force removal of bottom and bank material resulting from boat-induced currents.

d. **Channel-Meander** is a major cause of instability in riverine wetlands and is caused by current induced forces. The zone of attack is on the downstream portion of bendways and erosion is most severe at intermediate to high stages. Tractive force removal of material at the toe of the bank is the failure mechanism. The bank is undermined and fails due to loss of the geotechnical strength. The failed material is easily removed by the flow and the erosion cycle continues.

e. **Channel Braiding** is a cause of instability on streams having an overload of sediment or steep slopes where bars and islands can form producing a wide shallow channel. The bars and islands may function as wetlands. Erosion of banks, islands, and bars in braided channels occurs as a result of flow being diverted against the bank by the bars and islands. Zone of attack is highly variable and can occur at any position along the length of the channel. The mechanism leading to bank and island erosion is tractive force movement of the bank material.

f. **Ice and Debris** can reduce flow area and concentrate or deflect flow against otherwise stable banklines. The concentrated flow causes bank erosion through the mechanism of tractive force removal of bank material. The impact of ice and debris can gouge banklines, damage vegetation, and damage improperly designed protective measures.

g. **Water Level Fluctuation** is a cause of instability in riverine, depressional, and tidal wetlands and along reservoir shorelines. If water levels do not fluctuate, a distressed bank often erodes until it achieves a stable condition. As a distressed bank approaches a stable condition, a change in water level will disturb that condition. Water level fluctuations allow waves and currents to attack a bank at ever-changing elevations. Causes of water level fluctuations include naturally varying and controlled stream discharges, naturally varying and controlled lake levels, astronomical tides, seiches, wave setup, climatological effects, and navigation. A rapid drawdown in water level leaves banks in an unstable condition because the counterbalancing force of the water has been removed from the saturated bank and positive pore water pressures decrease the geotechnical strength of the bank. On steep banks, this imbalance in forces can cause a mass failure of the bank. On banks containing layers of different materials, rapid drawdown causes saturated banks to drain through the porous layers in the bank. Water movement through these porous layers may remove the porous material, leading to collapse of the bank. This failure mechanism is referred to as piping or sapping and is also found in riverine environments not having rapid drawdown but having saturated overbanks from ponds or poor drainage.

h. **Flow Constrictions** at bridge crossings, training structures, and floodplain encroachments cause local instability and result from the increased tractive force being able to erode otherwise stable banks.

Other causes of instability include rain splash, freeze-thaw, overbank drainage, and human activity. Overbank drainage and rain splash tend to be small-scale local instabilities but should be considered in designing protection methods. Freeze-thaw decreases bank soil strength which increases the potential for tractive force removal of the bank material.

General Design Requirements

After determining the causes and mechanisms of erosion, several general design considerations must be evaluated. As in the determination of cause, many of these considerations require someone experienced in their evaluation. Use the following considerations as a checklist to insure that important design factors are not overlooked.

Design Considerations

Geomorphology. Geomorphic evaluations involve determining the beginning point, ending point, and alignment of the protection. The protection may only need to extend along a limited reach of the threatened wetland. In other cases, the protection may have to be extended beyond the limits of the threatened wetland to insure adequate protection over the design life of the project. For example, in the riverine environment, a prediction must be made of the anticipated channel migration. In a typical meandering stream, migration of bendways is predominately in the down valley direction. Because of down valley movement, extending the protection far enough downstream is more important than the upstream limit.

For habitat and aesthetic reasons, less emphasis is given to a smooth alignment for wetland protection projects when compared to nonwetland protection projects. Accepting the existing alignment is almost always the case in existing wetlands. Created wetland boundaries are aligned for reasons of project function and economy. For example, a circular island shape was chosen for a proposed island wetland restoration project in the Chesapeake Bay to minimize the required length of shoreline protection for a maximum wetland area.

Ecological and Physical Barriers. Shoreline protection or erosion control alternatives should be designed so they do not inhibit the movement of organisms in or out of the wetland. For example, steep banks with crevices (e.g. riprap, block revetment) provide feeding areas for fish, but may trap small crustaceans and young animals. The selected alternative should not adversely restrict water flow in and out of the wetland. If the flow to the wetland is restricted, the change should not adversely affect water temperatures, dissolved oxygen concentration, and other concentrations of other chemical constituents. If the export of nutrients from the wetland is important, the alternative should not limit that transport.

Aesthetics. In a wetland environment, preservation of a natural appearance is important from both human and wildlife perspectives. Development and preservation of habitat and habitat diversity should be high priorities. Diversity of aquatic habitat is the result of diverse water depths and current velocities. Bank protection methods such as the indirect methods of dikes and groins promote diversity of aquatic habitat whereas relatively smooth revetments tend to reduce diversity. Rock structures and other bank protection methods provide stable substrate for macroinvertebrates. The impact of protection methods on aesthetics depends on the degree to which the protection measures are visually compatible with their surroundings. (Henderson 1986)

Hydraulic Setting. Quantifying the hydraulic setting is often required to determine the causes and mechanisms of bank erosion. Factors included in quantifying the hydraulic setting are outlined below.

Design Event. Identifying the design event is needed to select a protection method. The design event is the selection of event for which the erosion protection is designed. For example, erosion protection would be designed to allow a certain level of damage (or none at all) for a particular flood or storm event. The event is usually associated with a return interval, e.g. a flood with an expected occurrence of at least once in ten years, or once in 25 years. The conditions under which the protection should function needs to be identified to ensure an appropriate level of protection is designed.

While some of the protection alternatives presented here can be designed based on a selected design event, many lack the design guidance for such a design, mainly because they have not been extensively used or evaluated in the field. For these protection alternatives, demonstrated performance of the alternative under comparable conditions at a different site is the best guide.

When the major cause of erosion is wind waves or current forces, delineation of a design event is easier to determine than designs for causes like boat wave attack or debris damage. For the causes such as boat-induced erosion or debris damage, it is not possible to

determine a return interval for various event levels. Usually a less rigorous method is used, such as assuming a maximum wave height for the case of boat waves.

Water Level Fluctuations. Evaluation of water level fluctuation is needed to address potential geotechnical failures such as mass failure and piping. In the case of mass failure due to water level fluctuations, bank grading to a stable slope and protection of the slope against undermining is usually warranted. Piping-related failures are often addressed by grading to a stable bank angle, improving overbank drainage, and placing extensive filters on the regraded bank.

Wind Waves. Guidance for the estimation of wind-waves at a project can be found in the Corps of Engineers' Shore Protection Manual (SPM 1984). Another useful Corps of Engineers' tool for coastal engineering projects is the Automated Coastal Engineering System (Leenknecht et al, 1990). The system contains several programs useful for coastal engineering studies, including the estimation of wind waves.

The calculation of wind waves for a project requires information about wind speeds and directions, wind fetch, and water depth. In general, stronger winds, longer fetches and deeper water cause larger waves, and the wave direction is directly related to wind direction, as well as the influence of the water bottom. Values for these variables must be determined before wind waves can be predicted.

Local wind data should be reviewed to determine common or extreme wind events. A useful method is to categorize winds according to the number of occurrences for different combinations of wind speed and direction. Table 5-6 gives an example of such categorization of wind information. The wind speed and direction for which you want to estimate wind waves at the project site can be readily selected from the table.

Several corrections to measured wind data are usually required before they can be used for calculating waves. The corrections account for the elevation at which the wind was measured, whether it was measured over land, water or from a ship, and the temperature difference between the water and the air. Further, the wave estimation methods in the SPM require that wind speed values be converted to wind stress values. All of the corrections and conversions identified are outlined in the SPM.

Table 5-6
Percent Occurrence of Wind Speeds and Direction

MPH	North	NE	East	SE	South	SW	West	NW
0-10								
10-15								
15-20								
20-25								
>25								

To determine the general wave environment for the site, the corrections may often be neglected. However, the corrections should be considered if the wave calculations will be used in a final design. The equations presented below can be used for a quick assessment of wave conditions.

Calculating wave height and period by the SPM method requires that wind speeds be converted to wind stress. This can be done using the equation

$$U_A = 0.71U^{1.23} \tag{5-5}$$

where:

U_A = wind stress, m/sec
U = wind speed, m/sec

Assuming fetch-limited conditions, the wave height can be approximated as

$$\frac{gH}{U_A^2} = 0.283 \tanh\left[0.530\left(\frac{gd}{U_A^2}\right)^{\frac{3}{4}}\right] \tanh\left\{\frac{0.00565\left(\frac{gF}{U_A^2}\right)^{\frac{1}{2}}}{\tanh\left[0.530\left(\frac{gd}{U_A^2}\right)^{\frac{3}{4}}\right]}\right\} \tag{5-6}$$

where:

H = wave height, m
d = water depth, m
F = wind fetch, m
g = gravitational constant, m/sec^2

The wave period can be approximated as

$$\frac{gT}{U_A} = 7.54 \tanh\left[0.833\left(\frac{gd}{U_A^2}\right)^{\frac{3}{8}}\right] \tanh\left\{\frac{0.0379\left(\frac{gF}{U_A^2}\right)^{\frac{1}{3}}}{\tanh\left[0.833\left(\frac{gd}{U_A^2}\right)^{\frac{3}{8}}\right]}\right\} \tag{5-7}$$

where:

T = wave period, sec

The wind fetch is the extent of water over which the wind blows before reaching the shoreline of interest. On smaller bodies of water waves are usually fetch-limited, meaning the length of the fetch limits the size to which the waves will grow. But, on larger bodies of water

other conditions may apply. Waves may reach a fully developed condition for a given wind reaching their maximum size before reaching the end of the fetch. Another condition is a duration-limited condition where the wind does not blow long enough for waves to grow as large as they could for the given fetch. Also, when the width of the fetch has an effect on wave growth, the condition is called a restricted fetch condition. A restricted fetch condition can be identified by drawing radial lines out from the point of interest to the nearest obstruction along the radial line (for example, a shoreline across the lake). If the lengths of the radial fetches are significantly different from one another, the fetch may be restricted. A phenomenon of wave-growth over a restricted fetch is that the largest waves are often generated along a fetch that is not in line with the winds but rather in line with one of the longer fetch lengths. See Smith (1990) for a good description of the phenomenon and calculation techniques for predicting waves.

Generally, deeper water yields larger waves. For a given wind speed, direction, and fetch length, the wind waves generated on a deep lake will be larger than the waves generated in a shallow coastal estuary. However, waves will not be any larger once the depth exceeds a limiting depth no matter how deep the water is. Water depth also influences the form of waves to the point where they break on shore. As waves shoal (propagate into shallow water), their speed slows. Shoaling increases the steepness of the wave's profile, increasing the wave's height. When waves become too steep, they break. A simple rule is that the wave height will not exceed the water depth. That is, a wave with a height of 1 m would begin to break in a water depth of about 1 m.

Currents and Water Level Computation

Currents and water level computation in the riverine environment is beyond the scope of this handbook but an overview of the general concepts will be presented. The first step requires determination of a design discharge which comes from either computation of runoff from rainfall records and basin parameters or analysis of discharge gauging station records, if available. Gauging station data also provide information on the characteristics of the hydrograph (variation of discharge and stage with time) and the rate of rise and fall of the stream. Next, water surface profiles are computed using a design discharge in models such as HEC-2 (HEC 1990). Water surface profile computations on major rivers and streams are often computed by the US Army Corps of Engineers and the Soil Conservation Service. Local or District offices should be contacted for available information. The water surface profile computations provide the needed quantities of depth and average channel velocity for use in design of riverine protection. The design portion of this chapter will show how average channel velocity and depth are used to design the protection.

Vessel Effects are primarily a function of vessel speed, vessel shape and displacement, distance from vessel, and water depth. Regarding vessel shape and displacement, the two broad categories are commercial vessels, which are relatively slow but have large displacement, and recreational vessels, which are relatively fast but have small displacements. Commercial vessels rarely move fast enough to produce significant waves but they do produce substantial rapid drawdown when operating in confined channels. Methods for prediction of drawdown magnitude as well as other navigation effects can be found in PIANC (1987). Waves from recreational vessels decay rapidly with distance from the vessel and methods for their prediction can be found in Bhowmik et al. (1992). Bhowmik et al. (1992) measured wave heights on the

upper Mississippi River for uncontrolled runs during a busy holiday weekend and during controlled runs of various size recreational vessels and found a maximum wave height of 0.6 m.

Top Elevation of Protection. Wetlands, particularly in a tidal environment, have relatively low top bank elevation and protection often extends over the entire bank. In the riverine environment, many successful projects have been built with the top elevation of the structural protection well below the top-of-bank or design water surface. Bioengineering techniques are often used to protect the upper bank. Factors affecting the required top of structural protection in the riverine environment are stage duration, erodability of upper bank soil/vegetation, variation of hydraulic forces on the upper bank, bank slope, method of protection, and consequence of failure. USACE (1994b) provides a method for estimating the variation of hydraulic forces on the bank in the riverine environment.

Toe and Flank Protection. One of the most overlooked aspects in a bank protection project is consideration of the toe and ends of the design. Successful protection in both the tidal/coastal and riverine environments requires an evaluation of the potential for scour at the toe and ends. In the river, toe scour and the fact that many species of vegetation cannot withstand long term inundation are the primary reasons that bioengineering by itself will not provide stable bank protection. Some form of structural protection is often required at the toe of the river bank and must be able to withstand the changing bed elevation found in alluvial channels. In the wave environment, waves striking a hardened bank concentrate energy at the toe of slope that can result in scour and undermining of the protection. Procedures for estimating toe scour in the riverine environment are given in USACE (1994b) and in the wave environment in SPM (1984).

Once the scour has been estimated, there are two methods for providing scour protection. The first is to extend the protection down to the maximum estimated scour depth. This is often the preferred method in dry construction but becomes difficult and expensive when excavation is done underwater. The second approach is to place a flexible material that will adjust to the channel scour. This approach lets the stream do the excavation. Riprap is the most common material to use in flexible or “self-launching” aprons. Gabions and articulated concrete mattresses are also means of providing a flexible toe structure. Guidance for self-launching riprap and scour depth estimation in the riverine environment is given in USACE (1994b). In the wave environment, guidance for self-launching riprap is given in SPM (1984). Scour protection using gabions or articulated concrete mattresses should provide an apron length twice the anticipated depth of scour.

When considering the ends of the protection, it is desirable to terminate the protection in areas where the erosion forces are reduced. Unfortunately this is frequently not possible and the ends of the protection must be designed to not fail when the adjacent unprotected areas experience erosion. When using armor protection like riprap, increased layer thickness at the ends will allow the protection to adjust to minor adjacent erosion.

Geotechnical Setting Geotechnical design considerations include slope stability, filters, and subsurface drainage. Slope stability deals primarily with the stable bank angle which is a function of height, bank material, stratigraphy, stage fluctuation, groundwater conditions, and overbank loading. The purpose of filters and subsurface drainage is to control the movement of water and bank material beneath and through the protection. The indirect and sacrificial methods

presented later in this chapter generally do not require a geotechnical analysis of the bank being protected.

Surface Drainage. Surface drainage rarely causes failure of a protection alternative, but may cause maintenance problems, destroy vegetation, and damage the aesthetics of a site. The basic steps in preventing erosion from surface drainage are to (1) protect all bare ground unless slopes are flat and wavewash and runoff are moderate, (2) collect overland flow and wavewash, and (3) provide outlets to the river or open water body.

Filter Layers and Fabrics. Many protection alternatives require a filter layer between the protection-sediment interface. The filter layer's purpose is to prevent the filtering of sediments through the protection which would ultimately undermine or destabilize the protection. The filter layer can also distribute the weight of protection more evenly over the substrate. Filter layers are comprised of well graded gravel and stone. In many instances, a filter layer may be replaced by an acceptable filter fabric, the pores of which are specified based on the characteristics of the sediments. Additional information on filters is provided in Section 4.

Safety Factor. The consequences of failure must be considered. Only limited information is available for the design of many bank protection alternatives. Without well founded design information, determining a factor of safety is difficult and usually a conservative design is selected to compensate for uncertainties, or the design is based on convenience of construction, of materials, or some other feature. If the designer has experience with a particular protection alternative or the wetland region in question, then a factor of safety based on experience may be built into the design.

If the construction environment is difficult, or materials lack consistent quality, then as a factor of safety the design should account for the prospect of sections of below-average construction or low-quality materials. Safety factors can be reduced somewhat if inspection and maintenance are scheduled up front. That is, after a certain operating time if the project is showing signs of failure, remedial action could be taken to correct the problem.

Locally Available Materials. The cost of any project can be reduced if inexpensive locally available materials are used. A project design should always consider the advantages of using local materials in some element of the design.

Vandalism. Anticipate vandalism especially in areas where the public has access. Either employ hard-to-damage materials and designs or anticipate periodic maintenance. Consider using a protection scheme that will still work even if some portions of it are damaged. For example, some geotextiles tubes may be punctured by vandals allowing the contents to flow out. The decorative blocks of some concrete block revetments may be stolen. Removing the blocks compromises the system. Having an articulated concrete mattresses makes the blocks much harder to remove.

Educating the Public. Educating the public to the purpose of the project and making them feel a part of the project may help to reduce the frequency of vandalism and damage by people inadvertently working or recreating in the area.

Fate of Materials. Consider the possible fate of materials used in the bank protection design should the project fail or exceed its design life. Many geotextiles, tires, synthetic materials, metals, and treated woods do not degrade rapidly and may remain as an unappealing result of the project or pose a danger to humans and the environment.

Effect of Alternative on Local Waves and Currents. The impact of bank stabilization on areas adjacent to the protection must be considered in the design. In a naturally eroding stream system, bends migrate down valley. By stabilizing one portion of this system, the natural down valley movement is interrupted. The stabilized section causes the point of attack in the next downstream section to be fixed rather than transient. Depending on bank erodability and other factors, this constant point of attack can alter downstream erosion patterns and rates. Bank protection that significantly reduces channel area or deflects currents can increase downstream or opposite bank erosion.

Bank protection alternatives may influence the wave field near the protection. Waves refract and diffract near bathymetric variations and structures. For example, refraction will cause wave crests to bend around a mound of material on the bottom. The wave crest may bend so much that it collides with itself on the backside of the mound. The colliding wave crests can damage shorelines or protection works where otherwise the wave would have had no effect. Waves that pass by the end of a structure, such as a breakwater, will diffract into the region behind the breakwater and may cause unexpected damage.

Wave refraction and diffraction are complicated processes and questions about them should be referred to an expert.

Access. The selected bank protection should not adversely affect organisms requiring access to the wetland. For example, a riprap revetment may present a barrier to small or immature animals that must get from the wetland to open water. The voids between the revetment stones could act as a trap into which an animal may become trapped.

If the public has access to the site, the protection must not present a danger to them or their property.

Access also affects the selection of the protection method. In many wetland creation and restoration projects, land access for construction is not feasible for a variety of reasons. In many of these same projects, water access by large construction equipment is limited by shallow depths. Where larger construction equipment does not have access to the site, more labor-intensive alternatives may be required.

Animal Activity. Animal activity in and adjacent to protection methods can undo otherwise stable systems. Certain coastal crab species burrow passageways into exposed banks. These passageways, especially in conjunction with rapid drawdown from tows in confined waterways, can lead to a piping-type failure of the bank. Conversely, certain types of bank protection structures may create an ecological problem by preventing the use of the bank by such burrowing animals.

Another type of animal activity that has repeatedly caused problems is the consumption and destruction of vegetation protection systems by various animal species. Whenever vegetation is used in a protection plan, damage to the plants by animals and techniques to prevent it should be considered in the plans.

Water Chemistry. In tidal wetland environments, the tolerance of protection materials to seawater must be considered. Metals will corrode. Timber will rot. Certain species of vegetation will not tolerate certain levels of salinity (high or low).

Construction and Ease of Repair. Conditions can dictate the type of alternative selected because of limitations on the type of equipment that may be used. Conversely, costs must be increased to overcome poor construction conditions. If construction is expensive or difficult, then repairs or modifications will probably be costly as well. In any case, the ease of repair of the selected protection for the project of interest should be considered when designing the project and predicting future maintenance costs.

Navigation Hazard. If the public has access to the site by water, possible hazards to navigation from partially or fully submerged structures both offshore and on the bank must be considered.

Erosion Protection Alternatives

After determining the causes and mechanisms for bank erosion and quantifying the environmental setting of the wetland to be protected, a list of applicable protection methods can be developed. A combination of methods is often necessary to provide a desirable level of erosion protection. Table 5-7 presents common erosion control structures and their design requirements. The advantages and disadvantages of these alternatives are discussed below.

No Action

This alternative is selected when the environmental setting is mild enough not to require protection. However, natural systems are sometimes self-stabilizing. That is, they will erode until they form a more stable bank. It is possible, at times, to let an erosion problem continue if it appears that it will stabilize with time and that the wetland will not suffer unrecoverable losses. Experience and familiarity with the location is important in the somewhat subjective decision whether to take no action or apply protection.

Vegetation and Natural Materials

In low energy environments, well maintained vegetation and other natural covers often provide adequate erosion protection. Advantages and disadvantages of this alternative are summarized below.

Table 5-7
Erosion Control Structures and Design Considerations

Structure	Materials	Design Considerations
Riprap/ Revetments	Stone, cinder blocks, sand-cement bags	Riprap sized to resist stream velocities, wave impact and wash. Multiple layers may be required. Bank must be properly sloped. Good erosion resistance. Toe scour problematic when water depth is less than maximum wave height. Filter material behind revetment needed to prevent fine materials from being washed out from under revetment. Stone revetments - applicable to wide range of wave conditions with varying stone size. Cinder blocks, sand-cement bags limited to smaller breaking waves Riprap useful for all size projects and sites requiring immediate intervention Low to high cost: materials typically low cost if readily available in area, placement costs can be high, depending upon conditions and material requirements for hand placement and transportation. Reference Eckert, Giles and Smith 1978, Baker 1980, McCartney 1976, Binns 1986.
Windrow Revetment	Stone cobbles or graded fieldstone	A riverbank stabilization method utilized for high vertical banks. A trench is excavated parallel to river and filled with stone. The stone is covered with soil and the area reseeded. Over time, the bank may evolve to a steeply sloping configuration which will support vegetation. Aesthetically acceptable, natural looking stabilization method. Minimal disturbance to natural bank. Limited to relatively non-cohesive banks. Reference: USACE District Omaha 1980.
Tree Revetments	Green, felled trees	A streambank stabilization technique, trees are laid parallel to bank, overlapping 1/3 to 1/2 in shingle fashion. Trees are cabled to deadmen buried in the bank. Rock is placed along the bank with the trees. Certain species are more appropriate than others, having more branches and being less susceptible to attack by beavers (this will be somewhat site specific). Conifers and junipers work well. Recommended for small to intermediate streams free of ice. This revetment type provides excellent fish habitat. Reference: Binns 1986.
Gabions	Galvanized wire baskets filled with 4- to 10-inch-diameter rock and stacked and wired together	Maximum height approximately 10 feet with multiple thicknesses or counterforting (Eckert, Giles and Smith 1978) Susceptible to toe scour when placed on erodible substrate. Flexible gabion apron required. Moderate to high cost structure Reference: Eckert, Giles and Smith 1978, USACE 1977

(Sheet 1 of 4)

Table 5-7 (continued)
Erosion Control Structures and Design Considerations

Structure	Materials	Design Considerations
Articulated Concrete Mats	Interconnected concrete blocks	Used along steep riverbanks with moderate to high water velocities Reference: Way, Miller, Bingham and Payne 1992.
Geotextile Containers	Woven/ nonwoven permeable synthetic fabric/with or without impermeable liner	Utilized for underwater stability berms, breakwaters, sills, groins, breach and gully repair and scour protection. Bags filled with granular material or, if lined, with fine material through which vegetation may propagate Reference: Fowler and Sprague 1993
Groins	Rock, geotextile bags, gabions, piles and netting	A structure placed perpendicular to the shore to influence current direction and enhance sediment accretion Reference: Binns 1986, Abam 1993.
Floating and Fixed Tire Breakwater	Tires	Floating breakwaters can provide some wave attenuation, principally in low wave climates with short wavelengths. Floating tire breakwaters have been demonstrated to reduce wave energies by up to 80 percent. Materials to be protected must be somewhat scour tolerant: material retention is not a function of the floating breakwater. Floating breakwaters should be placed in water depths sufficient to float them at mean low water. Fixed tire breakwaters are threaded onto treated poles. Other materials could possibly be utilized for greater durability. Some turbulence and backwash will occur behind the breakwater Portable. Low cost materials. Assemblage and anchorage may be high cost. Reference: Eckert, Giles and Smith 1978, Knutson, Allen and Webb 1990, Markle and Cialone 1986.
Wood Breakwater	Timbers	A permeable structure, provides wave resistance by embedment in bottom. No deadmen or tiebacks required. Breakwater top located about 6 inches above normal high water level and slightly above elevation of marsh root mat Locate 20 to 50 feet offshore, about 10 feet seaward of lower vegetation limit, flatter slopes allow greater widths. Some sedimentation may occur behind the breakwater. Suitable sites have flat bottom and shallow water nearshore, less than 3 feet deep 50 to 100 feet offshore. Simple, nailed construction. Heavier construction may be required in high wake environments due to traffic at low tides. Small wood breakwaters not suitable for high energy environments Cost approximately one-third to one-half that of wood-end bulkheads Reference: Broome, Rogers, and Seneca 1992.

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Table 5-7 (continued)
Erosion Control Structures and Design Considerations

Structure	Materials	Design Considerations
Triangular Breakwater	Aluminum, tripod	A wave attenuation device. Flow -through design, stringers on front and back at 45-degree angle off the vertical. Fence cross sections are triangular. Vertical stabilizers extend into the sand bed. Reference: Ouzts and Machemehl 1977
Offshore Sill	Sand covered with stone, stone, gabions and sandbags, other suitable materials.	Sills reduce wave height by initiating wave breaking, protecting the land form behind the sill. Sill crest is typically at about low mean water, and 3 to 4 feet in width. Best located where tidal fluctuation is 2 feet or less (Eckert, Giles and Smith 1978). Low to medium cost structure. Placement costs may be most significant for sandbags and gabions requiring individual placement. Requires less material than a breakwater. Reference: Eckert, Giles, and Smith 1978, USACE 1977
Log Cribs	Timbers, coarse cobbles	Parallel rows of timbers are filled with cobbles. Log cribs are used to reinforce streambanks and are quite resistant to erosive flow, though not natural in appearance. Triangular log crib deflectors are used to deflect current at a point on the streambank. Susceptible to undermining. Reference: Binns 1986.
Christmas Tree Fences/ Branch Boxes	Brush or trees, fencing material	Christmas tree fences act to reduce water velocities and wave action and facilitate sediment accretion. Low energy environments - trees or brush can be tied into network of stakes Moderate energy environments - a crib is constructed of parallel rows of fencing material, posts and hogwire. This is filled with Christmas trees, or brush. The top of the cribs may be secured to prevent material from floating out. Greatest accretion occurs with fencing in shallow water sites (< 1 meter) tied in to shoreline. Other siting possibilities: closure of a breach, enclosure of a corner of a pond. Reference: Steller 1991, Allen 1992b
Slotted Wood Fences	Boards and pilings	A streambank stabilization structure. Pilings are driven at the toe of the bank and treated planks are bolted at intervals to the face of the pilings. Slotted fences reduce water velocities and induce sedimentation behind the fence. Utilized on bends with short radius of curvature and straight sections with high banks. Susceptible to undermining. Reference: Baker 1980.
Kellner Jacks	Long concrete beams or angle iron sections joined at centers to form hexapod	A streambank stabilization or training structure utilized to reduce flow velocities and induce sedimentation. Installed in single or multiple rows and anchored to pilings or deadmen. Used with slotted fences in some cases to further reduce water velocities. Reference: Baker 1980, Keown et al. 1977

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Table 5-7 (continued)
Erosion Control Structures and Design Considerations

Structure	Materials	Design Considerations
Bulkhead	Filled concrete culverts, post and timbers, aluminum sheeting and stone	A retaining wall separating land and water Reference: Coulombe et al. 1982.
Drop Structures	Earth, Soil-cement, Concrete Pipe, Riprap, Gabions, Sheet pile	Grade control structures to alter regime of stream from steep and eroding to stable. 1.5H to 1V embankment slopes possible with soil-cement structures. 2H to 1V or 3H to 1V typical drop slopes (Wulliman and Hanson 1990). Reference: Roberson, Cassidy, Chaudhry 1988, Watson and Abt 1993, Wulliman and Hansen 1990., Soil Conservation Service (SCS) 1984, SCS 1989.
Diversion Terraces/ Detention Ponds	Graded earth	Demonstrated in halting gully development. Utilized to divert and capture sediment transported in surface flow. Potential for use in protecting wetlands from excessive sedimentation from overland flow. Reference: Knighton 1984.

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Advantages

a. Vegetation and natural materials used for protection complement or become an element of the created and restored wetland.

b. Additional habitat can be created. Because the protection is often at the interface between open water and heavily vegetated water or land, it lies within the very productive portion of the wetland. Vegetated banks provide more appealing vistas for humans and more attractive habitat for wildlife that may otherwise be deterred by unnatural settings.

c. Vegetation is self perpetuating.

d. Vegetation will continue to strengthen and stabilize the bank assuming no destabilizing forces overcome the vegetation.

e. Successional or invasional species colonizing a site can add natural variety to the original protection scheme.

f. Vegetation minimizes the potential obstructions to the ingress and egress of organisms to the wetland, as well as the movement of water into and out of the wetland.

Disadvantages

a. Vegetation takes one to three years to fully develop.

b. Vegetation often requires stabilization measures to protect it during development.

c. Vegetation is only successful when applied to mild erosional climates.

d. Vegetation requires continuous monitoring and maintenance.

e. Minimal guidance is available for designing vegetation erosion protection based on wave and current conditions.

Common Reasons for Failure

a. Vegetation fails because there is no protection during the development stage.

b. Improper plant selection, handling, planting technique, or positioning may hinder development and propagation of plants.

c. The quality of the substrate was inadequate to support vegetation.

d. Opportunities to correct problems were missed due to inadequate monitoring and maintenance programs.

Design Characteristics

The following information was derived from Knutson and Inskeep (1982), Knutson and Woodhouse (1983), Allen and Klimas (1986), and Knutson et al. (1990). Other useful references are Coppin and Richards (1990), Gray and Leiser (1982), Schiechtl (1980), Allen and Webb (1993), and Allen (1992b).

Soil Type. Marsh plants will grow in a variety of soils from coarse sands to clays. The soil characteristics that affect the success of marsh plantings are **substrate stability, nutrient supply** and **ease of planting.** The substrate must be stable while the plants develop root systems to anchor themselves. Plants may need several weeks to develop an anchoring system within their roots. Soil nutrients are required to help plants grow, but the nutrient concentrations also determine how quickly plants will grow. When nutrients are readily available in the soil, plants will grow more quickly. Soils should be tested for nutrients and a determination made as to whether nutrients (fertilizers) should be added. Hard clays are more difficult than loose sand to plant and newly placed dredged material is usually too soft for planting machinery to be used. The labor involved in planting depends on the difficulty in penetrating the soil and whether hand labor or machinery is used.

Salinity. Important for coastal marsh plantings. Regularly flooded areas will have salinities at 35 parts per thousand (sea strength) or less. Vegetation can be found that tolerate these levels of

salinity. Most plants with a tolerance to seawater still tend to do better in areas where the salinity is below sea strength. Irregularly flooded areas or areas where waters do not flush significantly may have higher salinity values. Developing a successful vegetative stabilization scheme in these areas because of the stress on the vegetation is diffficult. If some doubt exists as to whether the vegetation will survive, locating existing stands of similar vegetation in similar (and nearby) areas will provide a good indicator for potential success.

Sunlight. Many emergent plants require maximum light exposure. For example, if emergents are planted near overhanging trees or bushes they will not survive. Water clarity and depth and duration of submergence during water level fluctuations affect the locations at which emergent vegetation will grow. Where an overhang is present, a recommended procedure is to clear the overhang back 3 to 5 m.

Shore Width Planted. Vegetation protects a bank by dissipating the nearshore current and wave energy. The broader the band of vegetation available for currents and waves to pass through, the greater the magnitude of dissipation. Based on observations, a practical planting width is 6 m. If the offshore slope is too steep to allow for this beach width, then the bank should be graded. (Grading should be done well in advance to allow for consolidation of sediments before planting.) Along the Atlantic coast where tidal fluctuations are less than 2 m, emergent vegetation can be found throughout the intertidal zone. Where the tidal fluctuations are greater than 2 m, establishment of vegetation in the lower part of the intertidal zone becomes more restrictive or impossible.

Sediment Supply. The loss of sediments from wetlands during storms or floods must be replenished during other periods. Sediment must be available through suspended or bedload transport. No guidelines are available for evaluating sediment supply.

Survival by Fetch. Survival of marsh plants is inversely proportional to the fetch length to the marsh. Knutson et al. (1990) found that 89 percent of projects exposed to less than a 2-km fetch were successful or partially successful (no erosion landward but some erosion at seaward edge of marsh). Conversely, 83 percent of the projects with fetches greater than 18 km were failures.

Survival by Shore Configuration. Projects located in narrow coves are exposed to waves only when waves are directed nearly perpendicular to shore. Projects located on headlands are exposed to waves from almost every direction. Hence, the likelihood of survival of a marsh in a cove is greater than that for a marsh established on a headland.

Survival by Inundation Duration. In the riverine environment, vegetation is generally unable to survive along the toe of the bank in streams having a continuous or base flow, or when there are strong hydraulic shear forces. For these reasons some form of structural protection is required up to the level at which vegetation can survive periodic inundation.

Sediment Grain Size. Sediment sizes in the intertidal zone are rough indicators of the wave-energy climate and so are potential indicators of planting success. Knutson et al. (1981) found that 84 percent of planted sites were successful or partially successful when the mean grain size was less than 0.4 mm. Conversely, they found that 82 percent of the sites failed where the mean grain size was greater than 0.8 mm.

Offshore Depths. Shallower offshore depths offer a better opportunity for successful shoreline stabilization projects. The shallower the offshore depths the smaller the wave heights for a given wind and fetch. For example, based on the methods in the SPM (1984) , a 13.9-m/s wind blowing over a 16-km fetch with a water depth of 1.5 m will generate a 0.5-m wave height. The same wind over the same fetch length but with a water depth of 12 m will generate wave heights of 0.8 m.

Tidal Currents. Marsh plantings as well as existing marshes are vulnerable to erosion by tidal currents. The proximity of tidal channels to the planting site and their tendency to migrate toward the plantings should be considered.

Plant Materials (sprigs, pot-grown, plugs). Sprigs are easiest to handle, transport and plant. They must be obtained from a field nursery planted at least one year in advance or from a nearby stable stand of vegetation. Pot-grown plants are easy to produce but cost 2 to 5 times that of sprigs to grow and plant. They are more difficult to transport and plant. Field-collected plugs are even more cumbersome to transport and plant and cost at least twice that of pot-grown plants.

Planting Methods (hand planting and tools, power-driven auger, machine planting). Hand planting with dibbles, spades and shovels is suitable for all types of plants. A power-driven auger is useful for difficult soils and for planting pot-grown seedlings and plugs. Machine planting is very efficient for large-scale plantings of sprigs and most can be equipped to handle seedlings.

Intertidal Plants. Several plants are available for stabilization projects. Two particular species are dominant in the intertidal zone. On the Atlantic seaboard and Gulf coast, smooth cordgrass (*Spartina alternaflora*) is dominant. On the West Coast, pacific cordgrass (*Spartina foliosa*) is more common. Notes on these emergent plants are provided in Table 5-4.

Riverine Plants. Plant species will vary up the bank depending on inundation frequency and duration and on their ability to withstand current attack. In the lower bank zone subject to frequent inundation, plants like reeds, rushes and sedges are recommended for planting. In the next zone up the bank grasses and woody plants are used which are flood tolerant and able to withstand inundation for up to several weeks. Various shrub-like willows such as peach leaf and basket are used in this zone. Alder and dogwood species have also been used in this zone. In the upper zone of the bank, inundation is much less frequent and grasses, shrubs and trees less tolerant to inundation are planted.

Other Considerations

Vegetation used for bank protection must often be protected itself (or its foundation enhanced) until it has had time to develop root systems and a thick stand. Vegetation is sensitive to its environment, such as water depth, water clarity, water quality, sediment type and nutrients. A good indication of whether vegetation will survive at a given location is to look for similar conditions in the region where vegetation has survived.

No significant design guidance on allowable velocity or wave heights is available for using vegetation for shoreline and streambank protection. The references provided at the end of this section are a good resource for available techniques, but quantitative design guidance is lacking

for designing vegetative protection for a given stream velocity or wave condition. A successful design depends on a thorough understanding of the physical and biological processes of the local environment and the protective vegetation and extensive experience with bioengineering techniques.

Vegetation can be protected initially by using some of the alternatives mentioned in the armor protection section below. Basically, anything that limits the intensity of waves and currents during vegetation development to an acceptable level is satisfactory. Additionally, you can apply techniques to strengthen the foundation in which vegetation is established. For example, natural or synthetic fiber mats can be sprigged with plants or the plants can be grown in fabric-sediment rolls. Vegetated mats and rolls can be grown offsite and transported to the project at construction. The mats and rolls provide a stable substrate for the plant roots. When vegetation is grown offsite, a significant lead time to construction is necessary so that the vegetation is ready for placement at the time of construction.

Vegetation has a limited range over which it is able to maintain sediment stability. That is, vegetation can only withstand a certain level of wave and current magnitude, before it is undermined or otherwise destroyed. However, even in cases where something other than vegetation is proposed for erosion control, you should always consider the possibility of adding vegetation to the design. For example, if rock revetment is necessary, it may be possible to plant vegetation between the rocks. The Natural Resources Conservation Service (NRCS) has established threshold levels for some vegetation protection techniques.

If a bank is too steep for vegetation to become established and thrive, the bank should be graded to an angle sufficient to support vegetation. If the bank previously contained vegetation but waves and currents have destroyed it and steepened the bank, then additional protection will be required after the bank is graded to prevent the bank from steepening again. The steepest bank slopes usually used for vegetation are 1:1. However, milder slopes of 1:2 or less are recommended.

Fiber Mattresses

Fiber mattresses consist of intertwined natural or synthetic fibers. The mattresses are porous allowing water to permeate while retaining sediments. Fiber mattresses do not require the use of a filter fabric or layer. Fiber mattresses are strong but depend on the quality of the materials for durability. The success of a mattress depends on its strength, durability, and the system used to anchor it.

Advantages

a. Biodegradable fiber mattresses can be used as temporary protection during the establishment of vegetation.

b. Vegetation can be sprigged in the mattress.

c. Mattresses are relatively inexpensive although, depending on the application, labor costs required to anchor the mattresses can be high.

d. Properly selected and installed mattresses are less noticeable and enhance aesthetic values.

Disadvantages

a. Mattresses are sometimes difficult to anchor sufficiently because the broad surface may experience large uplift forces.

b. If the anchoring system is damaged, the mattresses that are free to move may damage wetland vegetation and create an unsightly appearance.

c. No design guidance is available for proper selection of mattresses for given currents or wave conditions.

d. No guidance is available for sufficient anchoring techniques for given currents or wave conditions.

Common Reasons for Failure

a. Materials used in the mattress degrade too rapidly. For example, some glues used to hold the mesh together may soften in a wet environment.

b. Anchoring systems are pulled out by wave or current-induced uplift and drag forces on the mattresses.

c. Anchoring systems are undermined by currents or waves.

Design Characteristics

Mattress Overlap. A recommended technique for connecting horizontal edges of the mattresses is to overlap them high bank over low bank. The overlap should be at least one foot and the overlapping pieces connected together as part of the anchoring plan. If more than one length of mattress is required for a project the mattresses should be overlapped upstream to downstream. The mattresses should overlap by at least one foot and be connected as part of the anchoring system.

Mattress Thickness. Fiber mattresses are usually limited to 0 to 5 cm. thick. No guidance is available to determine an appropriate thickness for a mattress placed in a given current or wave climate. In general, a thicker mattress might be better as it keeps currents further away from the sediment interface and will tend to trap sediments moving in suspension or as bedload.

Other Considerations. Manufacturers may present the high stress that the material can withstand before it fails, but failure is usually due to things other than material failure due to stress. For example, a poorly installed mattress may be easily undermined by allowing water to flow freely beneath the mat. Depending on the velocity of water as it flows over the broad surface of the mat, uplift forces will be exerted on the mat. The anchoring system is most important under those conditions.

The durability of the product in the climate of interest is a very important consideration as well. Natural materials should be slow to biodegrade. Synthetic materials should also degrade slowly especially in the presence of sunlight. In coastal applications, the materials should be resistant to the chemical and corrosive effects of seawater.

Also to be considered are the potential effects if failure should occur. The movement and littering of mattress material about the region should not present any serious ecological, economic, social or political ramifications. For example, large sections of geotextile fabrics, nylon mesh and ropes, and other synthetic materials may be difficult or costly to locate and remove.

Cellular Concrete Mattresses (CCM) and Block Revetments

Erosion protection from wave attack or streamflow can be provided by man-made concrete blocks often labeled CCM for cellular concrete mattress. These interlocking or cable-tied blocks form a revetment similar to a gabion mattress. Cable-tied blocks are usually placed mechanically by crane and spreader bar, whereas interlocking blocks can be mechanically or hand-placed.

Most CCM blocks vary in size, shape, and thickness to accommodate velocities up to 7 m/s and wave heights up to about 2 m. A minimum thickness is about 10 cm. up to 20 cm. for all blocks with lengths and widths averaging about 35 to 45 cm.

Common to most concrete block or concrete mattress installations are these basic requirements:

a. Use a filter between the natural bank and the blocks.

b. Ensure the slope is stable under fully saturated conditions.

c. Ensure drainage of the soil to relieve hydrostatic pressures.

d. Ensure the CCM and block revetments are properly anchored.

e. The use of toe protection initially placed to the maximum scour depth is recommended, or a method that can adjust as scour occurs such as loose riprap or an apron of cable-tied CCM blocks.

f. Beware of inexperienced contractors and poor quality control. Attention to detail is critical with CCM blocks. Many of the failures that have occurred were the result of instability at the ends, edges, and transitions to other surfaces.

Advantages

a. The CCM open area of 20 to 25 percent allows colonization by vegetation.

b. The alternative can be cost-effective for urban streams.

c. CCMs are flexible and durable and can conform to minor bank settlement.

d. CCMs require less tonnage than riprap with a thickness roughly less than one-third that of riprap for channel flow applications.

e. CCMs are easily maintained and can be mowed to control vegetation if necessary.

f. The revetment voids and hardened substrate can provide habitat for various biota.

Advantages of Cable-Tied or Geotextile-Bonded Blocks

Blocks held together by cables or bonded to a geotextile to form a large flexible mat can have additional advantages over other types of block protection:

a. They offer greater flexibility, while retaining the advantage of interconnection to restrain blocks under extreme loading conditions.

b. They have a reduced risk of progressive local failure under extreme loading or deformation.

c. They are easily placed underwater and can be used as a flexible apron to address toe scour.

d. They can be placed rapidly including anchoring to subsoil.

Disadvantages

a. Cable-tied and geotextile-bonded systems are usually proprietary.

b. They can be expensive in rural areas.

c. They are susceptible to vandals removing blocks (non-cabled).

d. Presently lacking design guidance on some CCMs.

e. Unnatural appearance unless vegetation is allowed to hide protection.

Common Reasons for Failure

a. Toe scour undermining the revetment.

b. Excessive settlement leads to large hydraulic forces and an irregular block surface that can expose blocks.

c. Inadequate treatment and attention during design and construction to edges, ends, and transitions to other surfaces.

Design Characteristics

Block Type. Avoid designs where small concrete protrusions subject to breakage make up the interlocking design. Use appropriate concrete strength as opposed to construction blocks that are weak and friable.

Safety. Consider safety if public has access to the revetment. The submerged portion of the mattress can become slippery which may be more of a problem if the slope is too steep.

Block Size. Available guidance is found in the manufacturer's literature and may or may not be based on actual stability tests. Where guidance is not available for current attack applications, use a block thickness equal to one-third the riprap thickness computed from EM 1110-2-1601 (USACE 1994b).

Mattress Placement. Concrete mattresses come in assorted widths depending on manufacturer and placement equipment. The mattress is placed on the bank with the top edge buried behind the crest of the bank to prevent overwash overbank drainage erosion.

Toe Scour. The bottom of the mattress should be buried below the toe of the slope to the expected scour or, if using cable tied mattresses, should extend horizontally twice the expected scour depth beyond the toe of the bank.

Filter Layer. A geotextile fabric must be used behind the CCM mattress to prevent bank material from filtering through and undermining the mattress.

Bank Slope. In general, a CCM mattress should not be placed on a bank steeper than 1V:1.5H.

Riprap Revetment[1]

Riprap revetments are placed on a sloping bank and depend on the stability of the underlying soil for support. Fill material beneath a revetment must be adequately compacted prior to installation of the riprap. A riprap revetment, like the other revetments, consists of two or more layers. The first layer is a filter layer (or fabric). The filter supports the armor against settlement, provides drainage of groundwater through the revetment and prevents the retained soil from being washed through the armor layer by waves, currents, or groundwater seepage. The second layer is the armor layer which contains the larger stones that protect the filter layer and the bank. The armor layer maintains its position under wave and current forces either through the weight or interlocking of the individual units. Toe protection prevents displacement of the seaward or riverward edge of the revetment. Overtopping water must be controlled to prevent erosion problems at the top and behind the revetment.

[1] Much of the information in this section is taken from USACE (1981) (Low-cost shore protection, Section 54 study)

Advantages

a. Riprap is self-adjusting to small amounts of substrate consolidation or movement.

b. Riprap may experience minor damage and still continue to function adequately without further damage.

c. The rough surface of riprap dissipates local currents and minimizes wave runup more than a smooth revetment such as a concrete block revetment.

d. Material is readily available in many locations and can be less expensive than other structural alternatives.

e. Aquatic organisms can use the riprap as suitable habitat.

f. Riprap can be repaired easily by placement of additional stone when needed (if access to the location is reasonable).

Disadvantages

a. If material is not locally and readily available and easily transported to the site, costs can be prohibitive.

b. Riprap may present a barrier to organisms entering and leaving the wetland.

c. Riprap may not be aesthetically pleasing to some people.

d. Riprap may pose a hazard to people with access to the revetment.

Common Reasons for Failure

a. Flanking, overtopping and undermining of the revetment.

b. Settlement of sections of the revetment due to poorly consolidated substrate material.

c. Improperly designed or installed filter layer or fabric.

d. Undersized stone riprap displaced by large waves or currents.

Design Characteristics

Bank Slope. The underlying bank slope should be well consolidated and the steepest slope allowed should be 1 vertical on 2 horizontal (1:2) for waves and 1:1.5 for streamflow..

Filter Layer or Fabric. If a graded stone filter is used, it may be significantly more fine-grained than the armor layer. This may require the use of an intermediate layer of stone between the armor and the filter. This layer should consist of units about 1/10 the weight of stone in the

armor layer. If a filter fabric is used, this intermediate layer is also recommended because it acts as bedding, helps to distribute the armor stones' weight and protects the cloth from punctures and tearing under the weight of the armor.

Armor Stones for Waves. The longest dimension of individual stones should be less than three times the shortest dimension. Avoid using plate-like or cylinder-shaped stones. Stones should be angular and blocky, not rounded.

The size of stone can be estimated with the following formula from the SPM (1984):

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot \Theta} \tag{5-8}$$

where:

W	=	weight of an individual armor stone (N)
H	=	wave height (m)
S_r	=	specific gravity of the armor stone, unitless, = w_r/w_w
cot θ	=	slope of the structure expressed as horizontal units / vertical unit
K_D	=	stability coefficient (from Table 5-8)
w_r	=	unit weight of the rock (N/m^3)

If uniform quarrystone is used, the individual stones should range from 0.75W to 1.25W with 75 percent of the stones weighing W or more. For graded riprap, W corresponds to the minimum value of W_{50}, referred to as $W_{50(min)}$, and the recommended gradation is 3.6 W_{50} to 0.22 W_{50}.

Table 5-8
Stability Coefficients (K_D) for Stone Revetments

Armor Unit	K_D
Quarrystone	
Smooth Rounded	2.1
Rough Angular	3.5
Graded Riprap	2.2

Armor Stone for Streamflow. Guidance for riprap in streamflow applications is found in EM 1110-2-1601, "Hydraulic Design of Flood Control Channels," dated 1991 with Change 1 dated 30 June 1994. This guidance uses a procedure based on local depth-averaged velocity.

From EM 1110-2-1601 the equation for determining stone size is

$$D_{30} = S_f C_s C_v C_T d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5} \tag{5-9}$$

where

D_{30} = riprap size of which 30 percent (by weight) is finer, m

S_f = safety factor, unitless ($S_f \geq 1.1$)
C_s = stability coefficient for incipient failure, unitless
C_v = vertical velocity distribution coefficient, unitless
C_T = blanket thickness coefficient, unitless
d = local depth of flow, m
γ_w = unit weight of water, N/m^3
γ_s = unit weight of stone, N/m^3
V = local depth averaged velocity, m/s
g = gravitational constant, m/s^2
K_1 = side slope correction factor, unitless

Riprap thickness for most streambank protection projects is the greater of $1.0D_{100}$(max) or $1.5D_{50}$(max) and the blanket thickness coefficient (C_T) can be taken as 1.0. For riprap of this thickness and having a uniformity coefficient (D_{85}/D_{15}) between 1.7 and 5.2, the stability coeeficient for incipient failure (C_s) can be estimated as:

$C_s = 0.30$ for angular rock
$C_s = 0.375$ for rounded rock

The value for the vertical velocity distribution coefficient (C_v) should be:

$C_v = 1.0$ for straight channels or inside of bends
$C_v = 1.25$ downstream of concrete channels
$C_v = 1.25$ at end of dikes
$C_v = 1.283 - 0.2\log(R/W)$ for outside of bends (or 1.0 for R/W > 26)

where:
R = centerline radius of bend, m
W = water surface width at upstream end of bend, m

Recommended side slope correction factors (K_1) based upon slope are:

Slope	1V:1.5H	1V:2H	1V:3H	1V:4H or flatter
K_1	0.71	0.88	0.98	1.0

A minimum safety factor (S_f) of 1.1 should be used in all cases.

For bank protection $V = V_{SS}$ where V_{SS} is the depth averaged velocity at 20 percent of the slope length up from the toe. For natural channels typical of wetland applications, V_{SS} is determined from Figure 5-3 using average channel velocity, R, and W.

Toe. In the wave environment, the toe of the revetment should extend to one design wave height below the existing grade line to prevent undercutting. In lieu of deep burial, a substantial sacrificial berm or apron of additional rubble (with filtering) should be provided at the toe. See the alternative below “Windrow and Trench-fill Revetments and Toe Protection” for guidance

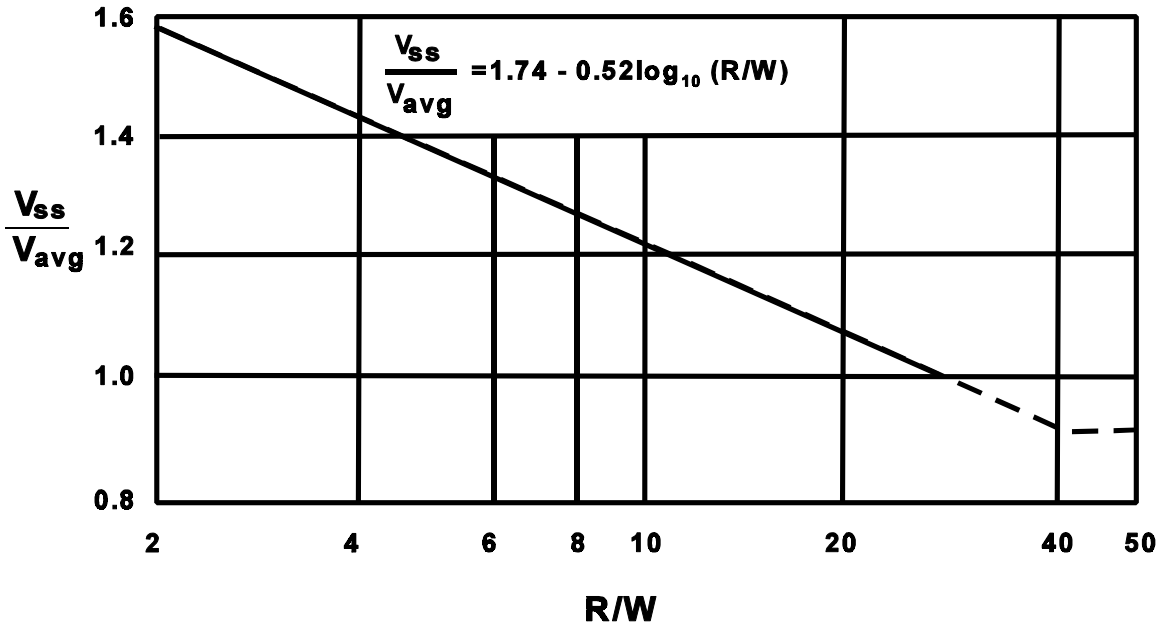

Figure 5-3. Depth averaged velocity as a function of R and W; V_{SS} is the depth averaged velocity at 20 percent of slope length up from toe.

for toe design in the riverine environment. In the streamflow environment, reference USACE (1994b) for toe protection guidance.

Dynamic Revetment[1]

A dynamic revetment consists of a larger volume of smaller stones as compared to a standard riprap revetment as described above. Because of the smaller stone size, the cross-sectional form of a dynamic revetment will be adjusted by the forces acting on it creating an equilibrium form. The larger volume of stone is required to ensure that the bank is fully protected even after the cross-sectional shape of the revetment is altered.

Advantages

a. Smaller equipment is required to place the smaller stones.

b. Placement of stone requires less care than a standard riprap revetment.

c. Smaller stone may cost less than larger stones required for a standard riprap revetment.

[1] This section is based on the description and design guidance presented in Ahrens and Heimbaugh (1989)

d. The final cross-sectional shape is more natural looking than a typical revetment. The composition and form is similar to that of a pebble or shingle beach.

e. The smaller stone sizes present less of an obstruction to smaller organisms that need to enter or leave the wetland.

f. The smaller stone sizes present less of a hazard to foot traffic.

Disadvantages

a. Smaller stones are not always less expensive than conventional riprap sizes. Without lower costs, the greater uncertainty in the performance of dynamic revetments (over conventional riprap revetments) may not be warranted.

b. Foot traffic and other activities may damage the equilibrium cross section obtained by the revetment. The cross section of a conventional riprap revetment is not likely to be damaged by foot traffic.

c. Less guidance and verification of guidance is available for dynamic revetments as opposed to conventional riprap revetments.

Common Reasons for Failure

a. The stone size is too small to remain stable under the given wave conditions.

b. Insufficient volume of stone is placed on the bank.

c. The stones are not constrained from moving laterally along the shoreline.

d. The revetment is undermined by poor filter layer, fabric, or overwash.

Design Considerations

Stability. The information presented here for determining stability should be used only for estimating the size and volume of stone required for a dynamic revetment. The references cited in this section should be consulted for a more detailed explanation for the appropriate use of the formulae provided.

Stone size can be related to wave height through the stability number given by Hudson and Davidson (1975) as

$$N_s = \frac{H_s}{D_{50}\left(\frac{w_r}{w_w} - 1\right)} \tag{5-10}$$

where:

N_s = stability number (lower value more stable), unitless
H_s = significant wave height, m
D_{50} = median stone diameter, m
w_r = unit weight of the stone, g/cm^3
w_w = unit weight of water , g/cm^3 (fresh water: 1.000 g/cm^3, seawater: 1.025 g/cm^3)

van de Meer and Pilarczyk (1986) used the stability number to classify structures and found that a dynamically stable rock slope has an N_s between 6 and 20. Therefore, given a value for H_s and selecting a value for N_s between 6 and 20, you can determine an adequate median stone size (D_{50}).

Ahrens and Heimbaugh (1989) found that the most significant volume of rock affecting stability is the volume above the assumed still water line (swl). Assuming constant cross-sectional shape, the volume can be considered as the area above the swl per unit width of shoreline, A. The best parameter to use to determine the potential success or failure of the structure is

$$A' = \frac{A}{H_s L_o} \tag{5-11}$$

where:

A = area above the still water line per unit width of shoreline, m^2
L_o = deepwater wavelength, m
H_s = deepwater significant wave height, m.

All designs should have a value of A' greater than 0.1 to prevent failure.

Gabions

Gabions are rectangular baskets or mattresses made of galvanized, and sometimes PVC-coated, steel wire in a hexagonal mesh, subdivided into approximately equal sized cells. At the jobsite, the baskets are unfolded and assembled by lacing the edges together with steel wire. The individual baskets are then wired together and filled with suitable diameter stone. The lids are finally closed and laced to the baskets, forming a large, heavy mass.

Advantages

a. Smaller stone used in a gabion can offer protection equivalent to the much larger stone used in a riprap revetment. (Assumes no destruction of the wire baskets.)

b. Can support some vegetation.

c. Can be cost-effective when using locally available stone filler.

d. Requires less tonnage than riprap. Gabion thickness is roughly one-third that of riprap revetment.

e. Flexible and durable if properly maintained.

f. Can be stacked to obtain near-vertical side slopes where available right-of-way is limited.

g. The gabion baskets can be built without heavy equipment.

h. Gabions are flexible and can adjust to minor settlement of their substrate.

i. Gabions can be repaired easily by mending or replacing damaged baskets and refilling them as needed.

Disadvantages

a. Wire mesh is subject to damage from strong waves, floating debris, corrosion, wear from high velocity sediments, and vandalism.

b. Labor intensive installation required.

c. Require monitoring and maintenance to identify wear before failure occurs.

Common Reasons for Failure

a. Baskets are not filled completely or adequately allowing them to move, resulting in abrasion and fatigue failures of the wire.

b. Baskets are damaged by floating debris, wear or corrosion.

Design Characteristics

Wire Characteristics. Wire is either galvanized or PVC coated for corrosive environments. See USACE (1993b) guide specifications for gabions.

Thickness. Expected flow velocities are required to determine the thickness of gabion structure required. Based on a smoothly graded bankline, design guidance in the Maccaferri Gabions publications (undated) suggests that gabion mattresses with thicknesses of 9, 12, and 18 inches will withstand velocities up to 10, 15 and 18 feet per second, respectively. Modular Gabion Systems guidance (undated) allows somewhat higher velocities. (Titles of undated gabion information publications are available from Maccaferri Gabions, Inc., Governor Lane Blvd, Williamsport, MD 21795: "Maccaferri Gabions; Gabions, Revet/Reno Mattress," "Maccaferri Gabions; Maccaferri Revet Mattress," "Maccaferri Gabions; Instructions for Assembly and Erection."

Placement. The normal gabion mattress width is 2 meters and common lengths are 3 or 4 meters with interior diaphragms on 0.7- to 1.0-meter spacings along the gabion length. Special mattress sizes can be fabricated for extensive protection projects. All mattress gabion revetments are tied together side-by-side to form a continuous blanket of protection. Gabion revetments, as any successful protection, must be constructed on a stable bank with proper internal drainage.

Bank Slope. Mattress-type gabions should not be placed steeper than 1V:1.5H but box-type gabions can be stacked where near-vertical side slopes are required.

Toe Scour. Gabion mattresses are an effective method of providing toe scour protection that will adjust when scour occurs. If placed horizontally, the mattress should extend out from the toe of slope a distance equal to twice the expected scour depth.

Filter. Geotextile fabric should be used beneath gabions.

Partial Bank Protection

On small to intermediate streams, most banks can be protected by a combination of structural protection on the lower bank and vegetation on the upper bank. As a general rule, the larger the stream, the greater the portion of the bank that must be protected with structures. Partial bank protection reduces the quantity of often costly structural protection and promotes vegetation in the riparian zone. An example is a willow post protection scheme that has performed satisfactorily in several applications. A minimal amount of structural protection such as riprap is placed along the toe of the eroding bank. The upper bank is graded to 1V:1H or flatter and 10- to 15-cm-diameter willow posts are augered vertically into the upper bank leaving 1.0 to 1.7 m exposed. The posts are sufficiently long to extend down into the water table to support the willow growth. The willow posts are placed along and up the eroding bank above the structural protection. While the willow and other vegetation is becoming established on the upper bank, the exposed posts provide flow resistance that reduces velocity on the upper bank. For high banks, the willow posts are often not long enough to extend to water that will support the willow, and other vegetation species are used on the higher regions of the upper bank.

Windrow and Trench-fill Revetments and Toe Protection

Windrow and trench-fill revetments are armor methods used in the riverine environment in which the stream erosion places the riprap revetment. Initially the riprap is placed behind the eroding streambank along the desired alignment in a trench at top of bank for windrow revetments and at mid to lower bank for trench-fill revetments. As the stream erodes back to the rock-filled trench, the rock falls or launches down the eroded slope and armors the bank. One of the primary requirements for successful stone launching is that the eroding streambank material must be relatively noncohesive so that the bank fails in a uniform manner. Cohesive banks fail in blocks and the rock-launching process becomes uneven and uncertain. Design guidance for windrow and trench-fill revetments can be found in USACE (1994b) and Maynord (1995).

Riprap is the most common method for providing toe protection in the riverine environment. The riprap is either placed down to the elevation of the maximum scour or, similar to windrow and trenchfill revetments, placed in a section of riprap called a weighted toe to launch down as toe scour occurs. The weighted toe method is particularly useful in protection constructed underwater. Volume of stone in the weighted toe is more important than the shape of the before-launch section.

Advantage

a. Eliminates underwater excavation.

Disadvantages

a. Requires greater stone volume due to uncertainty in the launch process.

b. Requires noncohesive bank to properly function.

Mild Offshore Slopes

In wave-dominated climates, a mild bottom slope (especially, when vegetated) is less likely to develop a serious erosion problem. A mild offshore slope helps to dissipate wave energy before it reaches the edge of the wetland. If the slope is vegetated additional energy losses occur. The slope must be made of material that can sufficiently resist breaking, propagating waves. If the wave climate is mild enough so that sand can be used, it will form reasonably submerged and subaerial stable bars and berms which help stabilize the sediment and dissipate wave energy.

A preliminary result of a test along the Gulf Intercoastal Waterway (GIWW), showed that a 1:15 slope fared better than a 1:10 slope for a dike exposed to boat wakes. After 2 years, the 1:10 slope shows an erosional scarp while the 1:15 slope does not. This result is not conclusive, however, as conditions offshore of the slopes have not been evaluated to determine whether they influenced wave conditions incident to the slopes.

Sand

In some wave-dominated projects where wave heights are small, if a sufficient amount of sand can be placed offshore of the wetland, it can act to cause waves to break and dissipate their energy before reaching the wetland. This is similar to the idea of developing very mild offshore slopes as mentioned above. If the sand is contained within the project area (bounded laterally by land or structures), then it may shift around within the region due to wave action and eventually form an efficient energy dissipation zone. A low “backwall” and lateral boundaries are present to keep the sand from moving out of the system. While no guidance or examples are available, it is worth considering that sand is used for shore protection on the open coasts which are exposed to the full fury of storms. Sand is a good energy absorber. If the wave climate and water levels are too great, however, sand may be pushed into the wetland area being developed, destroying the new or existing vegetation. A “back wall” is needed.

Sill

Sills are offshore structures with the crest usually submerged. The sill is designed to retain sediment and prevent it from migrating offshore. Design of a low-permeability structure is, therefore, important. A sill is often used in conjunction with other shoreward structures.

Berm

Submerged linear mounds of sediment may be placed offshore from the project site. The purpose of the berms is to reduce wave energy incident to the site by causing waves to break as they pass over the berms. No design guidance is available for constructing berms in very shallow waters near a wetland creation or restoration project. No guidance is available to determine the amount of wave energy reduction that will occur.

The berms will generally erode due to sediment transport by tidal-, wind-, and wave-induced currents. The rate of erosion and the resulting change in the berm shape is also unknown. The coarser the sediments used in the berm the more stable the feature will be. The advantages of using berms as part of the shore protection for a project are that they add interesting features and variations to local bathymetry, they afford (at least temporarily) some protection against wave energy, they add sediment to the local sediment transport system, and they provide a useful means of using otherwise excess sediment from a restoration or creation project. The disadvantages occur when the advantages do not apply. That is, berms are a disadvantage when they do not add useful variations to the local bathymetry but rather cover existing bathymetry, when they add too much sediment to the sediment transport system, and when they require significant effort to construct but do not survive long enough to provide much protection against incident waves. Berms should be used in conjunction with other alternatives for bank protection.

Stable Tidal Channel Design

Tidal wetlands must have a channel network that will provide the proper quantity of tidal water to achieve project goals of water quality and tidal circulation. While a channel network will form on its own in a created/restored wetland, initial sculpting of the wetland will cause channels to form in desirable locations that will meet project goals. The planform, channel cross section, and slope are determined by tidal characteristics, marsh sediment size, suspended sediment input, and biotic parameters such as the increase in stability that results from vegetation and the decrease in stability that results from burrowing animals. Because of the complex interactions of these variables, tidal channel dimensions are difficult to determine from first principles. Consequently, empirical methods are the best available method for small to medium tidal channel design in wetland restoration or establishment projects. The general form of these empirical relations are:

$$\text{Channel Width, Depth, or Area} = f(\text{tidal prism}) \tag{5-12}$$

This relation is similar to regime theory for upland channel design that is a function of discharge rather than tidal prism. Coats et al. (1995) provides empirical data and relationships applicable to California marshes and outlines a procedure to apply results to other areas.

Section 5
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