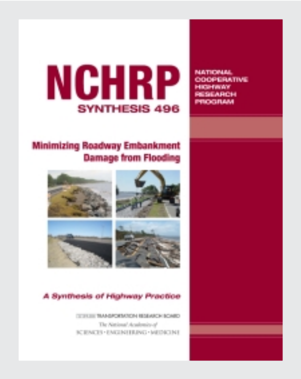

Minimizing Roadway Embankment Damage from Flooding

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Jean-Louis Briaud and Layal Maddah; National Cooperative Highway Research Program; Transportation Research Board; National Academies of Sciences, Engineering, and Medicine

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SYNTHESIS 496

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

Minimizing Roadway Embankment Damage from Flooding




A Synthesis of Highway Practice

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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CONSULTANTS
Jean-Louis Briaud
and
Layal Maddah
Texas A&M University
College Station, Texas

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Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

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Cover figures (clockwise from top left): Damage limited to the removal of performance turf and topsoil after armoring (*Courtesy*: Florida DOT); construction of stabilized roadway section (*Courtesy*: Florida DOT); damage from Hurricane Dennis (2005) along US-98, Franklin County, Florida (*Courtesy*: Florida DOT); installation of riprap placed over geotextile and then paved over (TH-9 south of Ada; *Courtesy*: Minnesota DOT).

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FOREWORD

Highway administrators, engineers, and researchers often face problems for which information already exists, either in documented form or as undocumented experience and practice. This information may be fragmented, scattered, and unevaluated. As a consequence, full knowledge of what has been learned about a problem may not be brought to bear on its solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem.

There is information on nearly every subject of concern to highway administrators and engineers. Much of it derives from research or from the work of practitioners faced with problems in their day-to-day work. To provide a systematic means for assembling and evaluating such useful information and to make it available to the entire highway community, the American Association of State Highway and Transportation Officials—through the mechanism of the National Cooperative Highway Research Program—authorized the Transportation Research Board to undertake a continuing study. This study, NCHRP Project 20-5, "Synthesis of Information Related to Highway Problems," searches out and synthesizes useful knowledge from all available sources and prepares concise, documented reports on specific topics. Reports from this endeavor constitute an NCHRP report series, *Synthesis of Highway Practice*.

This synthesis series reports on current knowledge and practice, in a compact format, without the detailed directions usually found in handbooks or design manuals. Each report in the series provides a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems.

PREFACE

Tanya M. Zwahlen
Consultant
Transportation Research Board

NCHRP Synthesis 496 is a state-of-the-practice report on how the transportation community is protecting roadways and mitigating damage from inundation and overtopping. In the absence of standard guidance, this report highlights major issues and design components specific to roadway embankment damage from flooding. It documents the mechanics of damage to the embankment and pavement, and the analysis tools available. The probable failure mechanisms are identified and various design approaches and repair countermeasures are highlighted.

The information presented in the synthesis is based on a review of the related literature, a survey of current practice, and a series of telephone interviews with state departments of transportation.

Jean-Louis Briaud and Layal Maddah, Texas A&M University, College Station, Texas, collected and synthesized the information and wrote the report. The members of the topic panel are acknowledged on the preceding page. This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As progress in research and practice continues, new knowledge will be added to that now at hand.

CONTENTS

Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the web at www.trb.org) retains the color versions.

MINIMIZING ROADWAY EMBANKMENT DAMAGE FROM FLOODING

SUMMARY

Roadway embankment damage from flooding is a shared concern among all U.S. states. Aside from the financial burden the states and federal government face to repair the damage, the preparation of an adequate design is a challenging task. This synthesis highlights major issues and design components in the absence of standard guidance specific to this topic. The information presented in the synthesis is based on a review of the related literature, a survey of current practice, and a series of telephone interviews. The probable failure mechanisms are identified and possible design approaches and repair countermeasures are highlighted.

The study presents a comparison between roadway embankments and levees to emphasize that embankments are not designed as flood control structures. The differences between riverine and coastal flood mechanisms are also stated because they impact the most effective way to minimize roadway embankment damage resulting from flooding. The common failure mechanisms in coastal and riverine environments identified by the survey results are overtopping, seepage (through seepage and underseepage), piping, wave action, softening by saturation, and lateral sliding over the foundation soil. Pavement failures and culvert-related failures are also outlined. Examples of failures and repair techniques are illustrated through 14 case examples gathered from six states that contributed information through the survey and follow-up interviews.

Based on the case examples, different failure modes were identified and different solutions were described. The effectiveness of the adopted solutions is usually dependent on the site conditions. A protection technique that proves successful at one site is not necessarily the adequate solution at other sites. To arrive at an adequate design, the following factors are generally considered: hydrologic and hydraulic factors, geological and geotechnical factors, legal and funding aspects, and risk. General observations and guiding principles are listed.

Hydrologic studies are essential in estimating the magnitude of the expected floods and in selecting a design flood. This information is used in nearly every aspect of the design. Hydrographs give information about the variation of the flow versus time. The use of hydrographs instead of peak flows can lead to more advanced analyses. Hydraulic methods use the hydrologic data to give an estimate of the water surface elevation, the overtopping height if overtopping is predicted, and the water velocity. Major geological considerations include the subsurface conditions, topography, floodplain and meandering potential, erosion and deposition, and basin characteristics and channel dimensions. Such considerations are essential in identifying the expected sources of damage at an early stage. Geotechnical calculations are crucial in designing against anticipated failure modes. Embankments are made of soil; therefore, identifying the characteristics of the embankment materials and their impact on the behavior of the embankment during flooding is very important. Key characteristics include the erodibility of the embankment materials, material properties (strength and permeability), and culvert- and pavement-related considerations.

In addition to the technical aspects previously described, the decision-making process is influenced by legal, regulatory, and funding aspects. After a failure, all decisions about the type of repairs as well as whether the same design will be repeated or whether betterments will be sought (temporary versus permanent, changing stream course, raising the freeboard) are bound by funding constraints, time constraints, constraints from interaction with other agencies (e.g., the U.S. Army Corps of Engineers, the Federal Emergency Management Agency, or EPA), and, in some cases, community constraints.

Available design methods to mitigate the impact of flooding on roadway embankments include the following:

1. Choosing the design flood
2. Overtopping
3. Seepage through the embankment
4. Seepage under the embankment
5. Wave erosion
6. Softening by saturation
7. Lateral sliding
8. Culverts
9. Pavements
10. Rapid drawdown.

The main countermeasures adopted by the surveyed department of transportation (DOT) engineers are identified, but their use is often best suited to a particular application. In the case of construction of a new roadway embankment, a key design element is the selection of an adequate roadway location, for which a thorough understanding of site characteristics and constraints (e.g., geology, geotechnical characteristics, stream hydraulic characteristics and meandering potential, upstream and downstream conditions, construction activities, and stormwater management facilities) is needed. In case of an old problematic site, it is essential to understand the key issue leading to recurring or aggravated damage. Possible issues include an increase in the severity of flooding events and the effects of stream instability caused by changes in upstream or downstream conditions (including man-made activities). By identifying all the major issues, an adequate design approach can be adopted. Such approaches would include one or a combination of the following options: relocating the embankment, stabilizing the stream, or designing for failure modes. The following protection techniques are commonly used to mitigate erosion derived from flooding mechanisms:

1. Vegetation
2. Riprap and geotextiles
3. Gabions
4. Articulated concrete blocks

5. Concrete lining

6. Paving.

The location and extent of the protection depends on the anticipated flow mechanisms.

It is essential to couple the design process and the use of countermeasures with engineering judgment, while keeping in mind all the issues outlined in this synthesis. It is also important to recognize that no design is foolproof and that the probability of failure is not zero. Consequently, it is also important to evaluate the probability of failure and the value of the consequence in terms of lives and economic loss. Ideally, it is the combination of the probability of failure and the value of the consequence or risk that can best guide design decisions, such as the selection of the design flood. For this purpose, future development of research relevant to risk factors and the level of risk accepted would be beneficial.

Future research studies are also needed to produce optimum design results. Such studies include relevant failure and success case studies, the impact of geomorphologic and geologic factors, efficient protection systems at different velocities of the water flow, guidance on the design and installation of culverts, the role of planning and management in the project success, and relevant software kits.

CHAPTER ONE

INTRODUCTION

INTRODUCTION

Roadway (highway) embankment damage caused by flooding in coastal and riverine environments is one of the current challenges faced by the United States. Aside from the economic hardships that result from repairing the damage caused by flooding, design difficulties related to the type and extent of protection against anticipated damage are commonly faced by departments of transportation (DOTs). Avoiding or at least minimizing embankment damage during flooding requires an understanding of the interaction between various geotechnical and hydraulic factors during extreme events. Relevant literature that describes such factors was collected to compile related technical information. DOT engineers were surveyed to identify the state of practice and to gather associated case examples. Follow-up interviews were conducted to complement that information. The findings are presented in this report.

Based on the findings, the extent of flood damage varies between the surveyed states, and so do the design approaches and remedial measures adopted by different DOTs. The components that would be considered in the decision-making process are highlighted throughout this report. In this introduction chapter, the adopted study approach is presented. Then the general differences between roadway embankments and levees are outlined, to avoid the false assumption that roadway embankments could function as levees. Next, the differences between riverine and coastal mechanisms are defined.

STUDY APPROACH

This study is based on a review of available literature and the information gathered from the survey responses and follow-up interviews with selected DOT engineers. This review is not intended to be a comprehensive study of all the relevant literature; rather it provides insights into the different aspects of the problem being studied. The information included pertains to studies carried out by such agencies as FHWA, U.S. Army Corps of Engineers (USACE), and NCHRP, in addition to some local and international technical literature. Information obtained from case examples mentioned in the survey responses and from the follow-up interviews are presented to reflect current practice. The survey questionnaire is included in Appendix A. An outline of the report is provided herein.

Chapter two discusses the common roadway embankment failure modes identified in the survey results and outlines possible mitigation measures. Chapter three presents examples of failure modes based on survey responses. It also summarizes the adopted protection and mitigation measures that were both successful and unsuccessful based on the limited information provided. Important concepts and considerations relevant to hydraulic and geotechnical disciplines are outlined in chapters four and five, respectively. A discussion of the hydrologic and geologic factors and their relevance to design are included. A brief discussion of the legal, regulatory, and funding aspects is presented in chapter six. Chapter seven describes current probability and risk practices and elaborates on relevant useful concepts. The survey results are summarized and presented in chapter eight.

The design steps that would minimize flood damage are compiled in chapter nine. Possible mitigation and maintenance measures are discussed in chapter 10. Finally, chapter 11 highlights ongoing research topics and future research needs.

ROADWAY EMBANKMENTS VERSUS LEVEES

The distinction between roadway embankments, levees, and other flood control structures was presented in a September 10, 2008, FHWA memo. This memo was issued in response to levee certification initiatives and to the “inadvertent or incorrect” designation of some roadway embankments as levees or other flood control structures that occurred during updates to Federal Emergency Management Agency (FEMA) maps. FEMA identifies flood risk areas through flood insurance studies. These studies are a part of the National Flood Insurance Program in participating counties. However, such studies were not carried out until much of the nation’s highway system was already constructed. As a result, some roadway embankments are located in flood plains. Roadway embankments and levees (Figure 1) are made up of soils and may look similar at first glance. However, they are generally different in many aspects, including the agencies responsible for issuing the relevant design document or guidelines, their purpose, the level of geotechnical analysis required, their design features, and the slopes and the fill materials used in their construction.

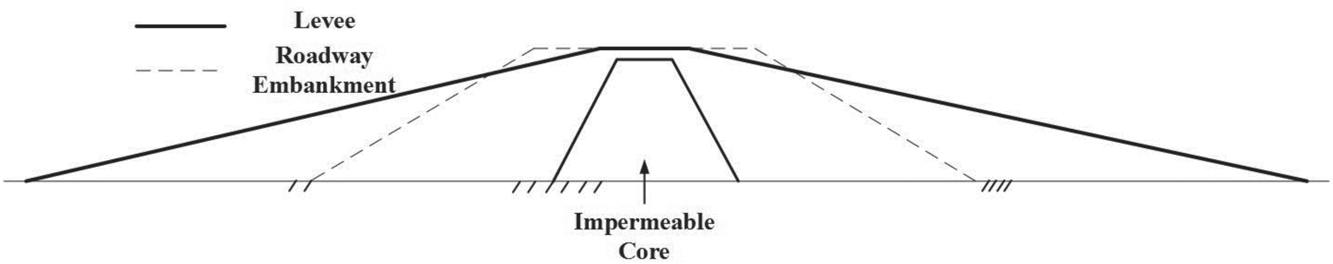

FIGURE 1 Roadway embankment versus levee typical sections.

This section highlights major features of such embankments versus those of levees.

The relevant documents and guidelines available to aid in the design of embankments and levees are presented herein. Such agencies as AASHTO and FHWA provide guidelines for the design of riverine encroachments. FHWA and USACE provide guidelines for coastal highway design. USACE has extensive technical guidelines widely used for levee design.

The purpose of a roadway embankment is to safely carry traffic as a part of the highway network. A levee, on the other hand, is a flood control structure designed to protect urban or agricultural areas from the anticipated design floods (USACE 2000). Although roadway embankments that lie within the floodplain are prone to similar hydraulic forces as levees during flooding events, they are not generally designed to retain water during flooding. As a result, the level of analysis required varies between the two structures. In the case of roadway embankments, the level of analysis is determined by highway location and project requirements. In the case of levees, the analysis is more specific and focuses on preventing the overflow of water to the dry side of the levee up to design flood levels. As a result, the design features, adopted slopes, and fill material used in construction vary between the two structures.

For example, levees sometimes feature impermeable cores while roadway embankments do not. Also, levee embankment slopes are generally flatter than those used in roadway embankments (Figure 1). In addition, gradation is specifically considered in levee design, to prevent seepage and erosion. Table 1 presents a comparison between roadway embankments and levees.

Because roadway embankments could lie in riverine or coastal environments, the differences between riverine and coastal mechanisms and flooding duration are presented in the next section.

COASTAL VERSUS RIVERINE EMBANKMENTS

Roadway embankments within the floodplain of a river are termed riverine embankments. These embankments are subjected to hydraulic forces from flooding rivers or run-offs that occur during flooding events (Figure 2*a*). Coastal roadway embankments are raised earth structures located in coastal environments and on which the pavement structure is constructed (Figure 2*b*). These embankments are subjected to coastal processes that include wave action and surges resulting from hurricanes. The terminology shown in Figure 2 will be adopted throughout the text. The difference between the riverine and coastal mechanisms and flooding durations are presented herein. Additionally, important design parameters for embankment design in both environments are highlighted.

TABLE 1
COMPARISON BETWEEN ROADWAY EMBANKMENTS AND LEVEES

	Roadway Embankments	Levees
Agencies That Issue Relevant Documents/ Guidelines	AASHTO/FHWA	USACE
Purpose	Carry traffic as a part of the highway network	Flood controlling/retaining structure
Level of Analysis	Dependent on highway location and level of geotechnical analysis required	Specific focused type of analysis
Design Features	No internal impervious core-homogeneous materials	Sometimes internal impervious core (zoned)
	Freeboard varies based on considerations related to state requirements and/or regulatory agency requirements (chapter six)	Freeboard included
	Wider crest to accommodate anticipated traffic (multiple lanes)	Narrow crest (about 3 m)
Slopes	Steeper slopes: 1H to 1V, 2H to 1V, 3H to 1V, and 4H to 1V	Relatively gentler slopes: 5H to 1V and 6H to 1V
Fill Material	Could be homogeneous embankment consisting of relatively similar/uniform material or heterogeneous depending on available material (especially for local systems such as county roads)	An inner core (impermeable materials) is introduced
	Gradation generally not designed as a barrier against water	Gradation selected to prevent seepage, piping, and infiltration

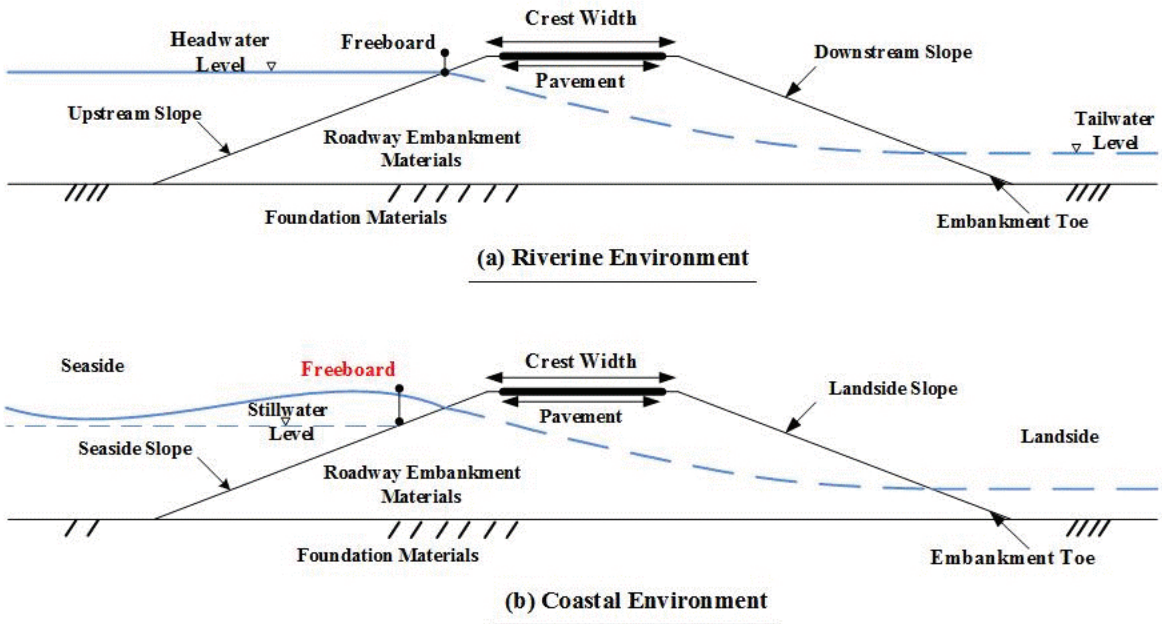

FIGURE 2 Typical roadway embankment section in (*a*) riverine and (*b*) coastal environments.

Coastal Versus Riverine Mechanisms

Coastal environments exhibit flow conditions in which factors such as tides, wind-generated waves, and storm surges play an essential role. Waves and tides represent an unsteady, periodic, pulsating, and intermittent flow component. Surges present a relatively steady motion for a short time with respect to inland flow discharges. The design water surface elevation mainly involves the estimation of the tidal water elevations and the wave heights. The tidal and wave changes are not typically sensitive to the presence of an embankment. Rivers, on the other hand, are relatively more sensitive to the presence of an encroachment or an embankment. For example, changes of conditions downstream could affect the upstream conditions. This might alter erosion and deposition patterns or jeopardize the river stability.

On the other hand, rivers commonly show relatively constant flow discharge over a certain period of time. In other words, rivers generally do not exhibit sudden changes of flow overnight. Also, river waves do not generate run-up distances along the riverine embankment upstream slope as large as their coastal counterparts do along the seaside slope of a coastal embankment. Further information can be found in references that include *PDDM Manual* (FHWA 2014), *HEC-25* (Douglass and Krolak 2008), USACE (Hughes 2008), Chen and Anderson (1987), and Clopper and Chen (1988). A summary of the differences between coastal and riverine overtopping mechanisms is presented in Table 2.

Coastal Versus Riverine Flooding Duration

River floods may last for days, and sometimes weeks. The survey results revealed that Minnesota, for example, experienced overtopping during the Red River flood of spring 2011 that lasted 3 weeks. Based on information gathered in a previous study by Chen and Anderson (1987), sites in Arkansas and Missouri (December 1982 flood), Wyoming and Colorado (May 1983 flood), Arizona (September 1983 flood), and Wyoming (August 1985 flood) experienced overtopping durations that ranged from 9 hours to more than 3 days. As revealed in case examples from Minnesota and Colorado in chapter three of this report, one factor that affects the duration of flooding is the area's topography. In Minnesota, longer flooding (and possibly inundation) durations occur in flatter terrains (Oslo) than in steeper terrains (Ada). In cases of steep terrains, such as those in Colorado, shorter duration and higher velocities are anticipated.

TABLE 2

COMPARISON BETWEEN COASTAL AND RIVERINE OVERTOPPING MECHANISMS

Characteristics	Coastal Environments	Riverine Environments
Nature	Unsteady discharge/periodic/pulsing/intermittent flow	Steady discharge/continuous flow
Occurrence	Wave overtopping can occur even if still-water level ≤ embankment height	Overtopping occurs if headwater level ≥ embankment height
Duration	Relatively short (hours)	Relatively long (days)
Velocities	Peak velocities sustained only for a brief time	Peak velocities could last for prolonged durations
Erosion Mechanisms	Intermittent and consistent with periodic/intermittent flow	Continuous erosion process that decreases with steady decrease of velocity
Erosive Capability	Wave action more prominent	Long flow duration

Coastal highways, on the other hand, may be subjected to hurricanes and the associated ocean surges and waves that may overtop an embankment for a relatively shorter period; that is, a couple of hours. The resulting damage, however, could be greater than that of rivers because of the higher velocities encountered.

Design Parameters

The main parameters involved in the hydraulic design calculations of roadway embankments are the peak discharge, expected flood height, and hydrograph (discharge as a function of time). In coastal environments, wave parameters such as the run-up elevation and the significant wave height are also included. The hydrologic and hydraulic parameters are described in chapter four.

From a geotechnical perspective, if no protection systems are used, the key parameters are related to soil type, compaction level, soil strength, and erodibility. When slopes are protected, the embankment material is designed to support the weight of the applied protection system. Precautions related to the type and extent of protection are typically taken to prevent leeching of the underlying material through the surface protection. These precautions are discussed further in chapters five and 10.

The design process would ideally include the following:

- Selection of a design height (including freeboard) and the slopes of the roadway embankment
- Estimation of the overtopping duration and the depth of overtopping measured from the embankment crest if the embankment is designed to overtop
- Velocity calculations to select and design an adequate protection system
- Evaluation of embankment damage given an overtopping duration.

SUMMARY

In this chapter, the study approach was outlined. The differences between embankments and levees were discussed. A comparison between riverine and coastal roadway embankments was also presented. The next chapter will explore the different failure modes that generally occur in embankments. As discussed in later chapters, it is important to match the failure modes that frequently occur with the practices available in order to mitigate their effect.

CHAPTER TWO

FAILURE MODES

INTRODUCTION

This chapter uses the survey results and literature review to present roadway embankment failure modes commonly encountered during flooding. These failure mechanisms involve hydrostatic and hydrodynamic forces that result from overtopping, seepage forces, and the lateral pressure caused by headwater elevation. Overtopping forces can lead to surficial erosion. Seepage forces can lead to saturation, internal erosion, and piping. Lateral pressure can cause the embankment to slide on its foundations. Relevant design parameters and available calculation methodologies will be discussed further in chapters four and five.

The following references pertain to levees and other flood control structures; in some cases, the structures exhibit failure mechanisms similar to embankments.

- Chen and Anderson (1986), *Methodology for Estimating Embankment Damage Due to Flood Overtopping*
- Chen and Anderson (1987), *Development of a Methodology for Estimating Embankment Damage Due to Flood Overtopping*
- Clopper and Chen (1988), *Minimizing Embankment Damage During Overtopping Flow*
- Richardson et al. (2001), *River Engineering for Highway Encroachments*
- Seed et al. (2005), *Investigation of the Performance of the New Orleans Flood Protection Systems in Hurricane Katrina*
- Douglas and Krolak (2008), *Highways in the Coastal Environment, HEC-25,* Vol. 1
- Bonelli et al. (2013), *Erosion in Geomechanics Applied to Dams and Levees*
- Douglas et al. (2014), *Highways in the Coastal Environment: Assessing Extreme Events, HEC-25*, Vol. 2.

COMMON FAILURE MODES

As part of the survey, DOT engineers were asked to rank failure modes based on the frequency of occurrence in their state from the most common (5) to the least common (1). The ranks were summed for each failure mode to produce a score. The modes are presented based on their scores from highest (most frequently occurring) to lowest (least frequently occurring):

1. Overtopping erosion
2. Softening by saturation
3. Underseepage and piping
4. Through-seepage (internal erosion) and piping
5. Wave erosion
6. Lateral sliding on foundations
7. Other failure modes including culvert failures and pavement failures.

Each of the failure modes listed ranked as most frequent in at least one state (Figure 3). Other failure modes, including failure related to culvert clogging or erosion and pavement failure, are also discussed.

OVERTOPPING

Overtopping of roadway embankments occurs when the headwater (still water or waves for coastal environments) reaches the point on the embankment crest with the lowest elevation. Overtopping flow mechanisms can occur in coastal and riverine environments as follows:

Case I: Overtopping resulting from an increase in a river's headwater level as it reaches the crest (Figure 4*a*). As identified by Hughes (2008), USACE states that this flow mechanism can also occur in coastal environments if the still-water level slowly increases with time as a result of tide action, time-varying hydrograph, or long-period seiching (the development of a standing wave in an enclosed—e.g., lakes or reservoirs—or partially enclosed—e.g., harbors or seas—body of water).

Case II: Overtopping resulting from wind-generated waves, even though the still-water level has not quite reached the embankment crest (Figure 4*b*). This case is most prominent in coastal environments.

Case III: Overtopping resulting from a combination of Case I and Case II (Figure 4*c*) where the still-water level exceeds the embankment crest elevation and wind-generated waves provide a pulsing, unsteady component to the over-

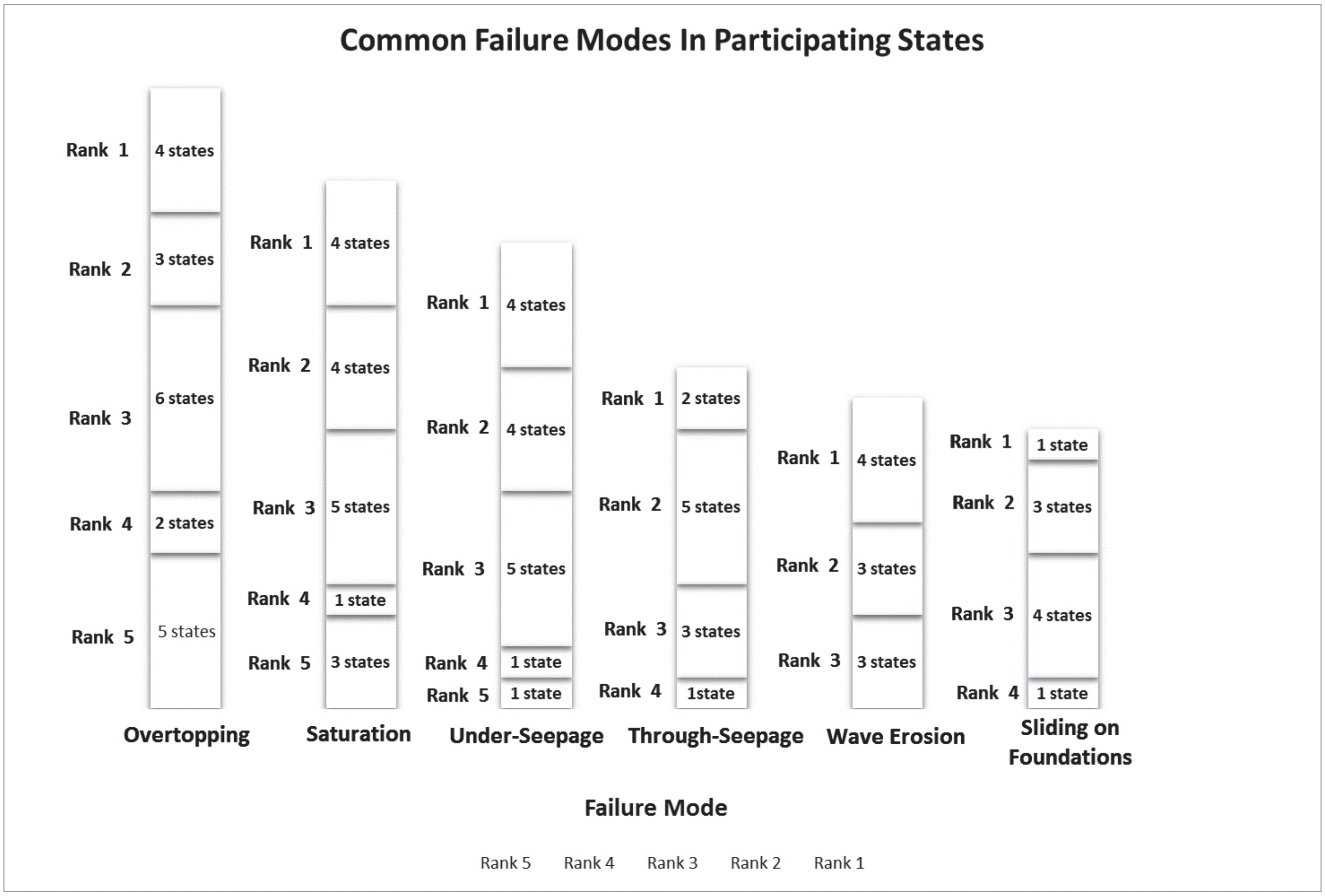

FIGURE 3 Common failure modes in participating states.

topping flow. This case is the most problematic, and generally occurs during hurricane events in coastal environments. These mechanisms are further explained in the following sections based on available studies (generally limited to Case I).

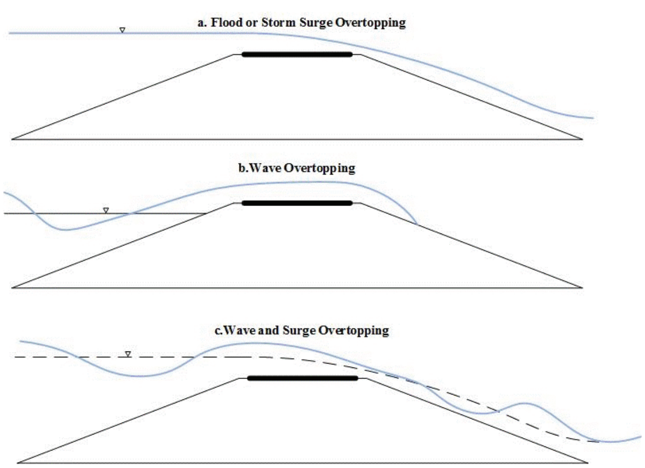

FIGURE 4 Overtopping mechanisms (modified after Hughes 2008).

Flow Patterns

The flow patterns associated with overtopping can take different forms. Kindsvater (1964) differentiates between them as follows: (1) free plunging, (2) free surface flow, and (3) submerged flow. The development of these flow patterns depends on the tailwater condition, but the most common is free surface flow. A free plunging flow occurs when the falling water produces a submerged hydraulic jump on the downstream slope under the surface of the tailwater. A free surface flow occurs when the water follows the contour of the downstream slope rather than plunging into it. A submerged flow occurs when the depth of water on the downstream side rises and the discharge becomes controlled by both the upstream and downstream heads.

In the most common case of free surface flow, three overtopping flow zones can be identified (Clopper and Chen 1988; Figure 5):

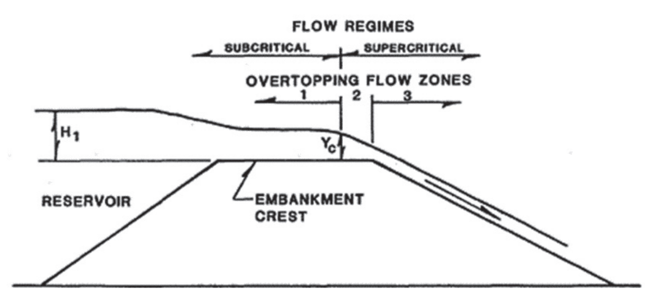

FIGURE 5 Hydraulic flow regimes and overtopping flow zones (after Clopper and Chen 1988).

- Subcritical flow over the embankment crest (Zone 1), which is characterized by the critical depth (Y_c) that is approximately equal to two-thirds of the overtopping head (H_1). The water surface elevation is drawn down by one-third H_1 because of the velocity head. The location of the critical depth is dependent on the embankment geometry and the overtopping head.
- Supercritical flow over the embankment crest (Zone 2), which occurs across the stretch of the crest downstream from the critical depth. The corresponding velocity is a function of gravitational acceleration (g) and the critical depth (Y_c).
- Supercritical flow on the downstream embankment slope (Zone 3) which occurs along the downstream slope of the embankment. It is described as supercritical owing to the steep downstream embankment face and turbulence from the surface roughness. Corrections are made for the slope angle in flow depth calculations and for air entrainment in the unit weight of the air entrained water. The corrected parameters are then used in relevant velocity and shear stress calculations.

Another factor that could be considered in design, particularly for protection systems, is the zone of negative atmospheric pressure that may develop as shown in Figure 6 (Clopper and Chen 1988). This pressure develops on the downstream embankment slope and could lead to separation between the protection technique and the embankment surface if not included in the design considerations. This will be further discussed in chapter four.

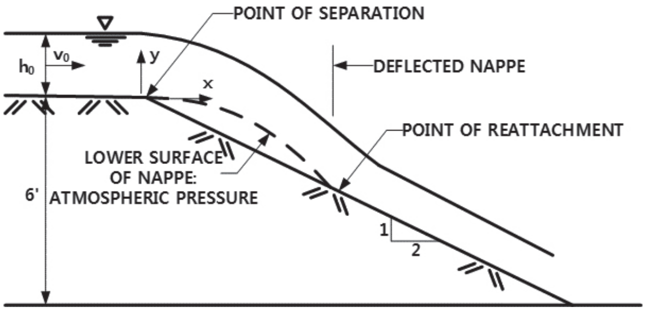

FIGURE 6 Profile of water nappe and theoretical subatmospheric pressure zone (after Clopper and Chen 1988).

Erosion Modes

Erosion mechanisms depend on the flow characteristics. Chen and Anderson (1987) differentiate between two mechanisms as follows:

- **Freefall condition:** In this case, erosion starts at the embankment toe and proceeds upstream in a head-cut as shown in Figure 7. In time, this failure mode would lead to a serious breach because the head-cutting at the toe affects the overall embankment stability and induces erosion losses.
- **Submerged condition:** In this case, erosion begins at the downstream slope and propagates both upstream and downstream as shown in Figure 8.

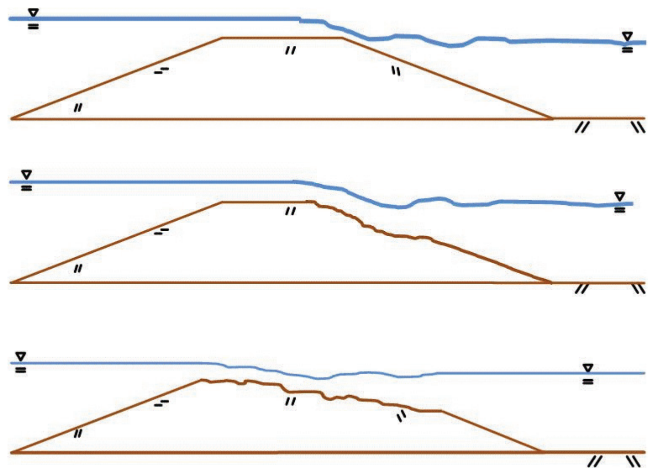

FIGURE 7 Progressive stages of unprotected embankment erosion under freefall flow condition (after Clopper and Chen 1988).

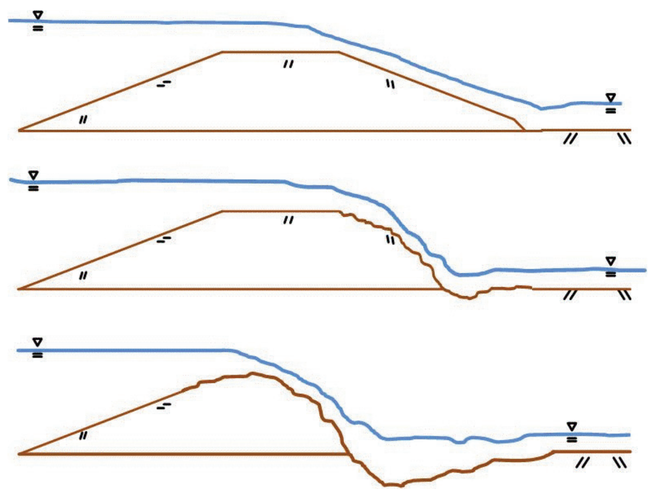

FIGURE 8 Progressive stages of unprotected embankment erosion under submerged flow condition (after Clopper and Chen 1988).

Damage from overtopping can be mitigated by using the different protection and mitigation techniques discussed in chapter six. In general, the downstream (landward) slope would be protected against overtopping by either waves, continuous flow, or a combination of waves and continuous flow. In coastal environments, both the seaward and landward slopes would have to be protected because erosion can take place on the seaward face as the storm recedes.

Mitigation Measures

Douglas and Krolak (2008) present a number of mitigation strategies for coastal roads that over-wash. These strategies include adequate selection of the road location, consideration of the road elevation, construction of sand dunes, and armoring of shoulders to prevent back erosion. For both riverine and coastal embankments, different protection options for the downstream slope are discussed further in chapter eight.

References that describe the design and applicability of different protection options on the downstream slope of overtopped embankments include *HEC-23* (Lagasse et al. 2009a and b), *NCHRP Report 568* (Lagasse et al. 2006b), *HEC-14* (Thompson and Kilgore 2006), HDS-6 (Richardson et al. 2001), and *HEC-11* (Brown and Clyde 1989).

SOFTENING AS A RESULT OF SATURATION

Saturation occurs when water seeps into the embankment, yet it is not associated with erosion that leads to a breach. Saturation can occur if the embankment is subjected to prolonged durations of rainfall or flooding. The saturation impacts the effective stress within the soil that makes up the embankment. The effective normal stress σ` is given by

$$\sigma` = \sigma - au_w \qquad \text{(Eq. 1)}$$

Where:

σ is the total normal stress on the same plane in lb/in^2,

α is the water area ratio often taken equal to the degree of saturation as a first estimate, and

u_w is the water stress in kPa (psi).

The effective stress directly impacts the strength and compressibility of the soil because it directly relates to how hard the soil grains are pushing against each other. Several cases can occur:

1. If the embankment soil has a degree of saturation less than 100%, the soil is unsaturated and the water in the soil pores is in tension. This tension results from the attraction between water and the minerals making up the soil particles. Any wetting of the soil will increase the degree of saturation, decrease the water tension, and decrease the effective stress.

2. If the embankment soil has a degree of saturation equal to 100% but the soil is saturated by capillary action, again the water in the soil is in tension. Any wetting of the soil will increase the water content, swell the soil, decrease the water tension, and decrease the effective stress.

3. If the embankment soil has a degree of saturation equal to 100% and the soil is partially underwater, the water is in compression. Any further rise in the water level will increase the water compression and decrease the effective stress.

Any one of these cases will lead to a decrease in strength and a decrease in soil stiffness, which is exemplified by the following example of the impact of water content and associated water tension on the modulus of a soil during a Proctor Compaction Test. A standard compaction test was conducted at different water contents on a silty sand. For each water content, the dry density was measured and the modulus was determined with a tool called the BCD (Briaud 2013). As Figure 9 illustrates, the drop in modulus is very significant on the wet side of the optimum water content, while the dry density does not reflect this drastic loss of stiffness. Such loss of stiffness will lead to increased compression and deflection of the pavement under traffic loading.

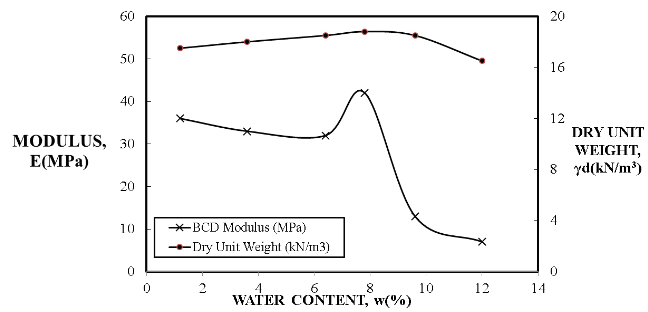

FIGURE 9 Impact of water content on soil modulus (Briaud 2013).

After the flood ends and as the water level recedes, another issue can arise. If the embankment soil is a fine-grained soil, the water stress inside the embankment may remain locked in at the flood condition level for some time. In this case, the shear strength of the soil remains low as it is the one associated with the low effective stress. But since the water level dropped, the horizontal water pressure on the upstream side is no longer there to provide lateral support to the upstream slope. This is called a rapid drawdown condition and it is the worst condition for the stability of embankment slopes (Chen and Anderson 1987).

UNDERSEEPAGE

Underseepage can occur particularly if the foundation material supporting the embankment is pervious. Underseepage results from the difference in total head between the two sides of the embankment and is likely to occur if the embankment itself is less pervious than the foundation soil on which it rests. In this case, it is easier for the water to travel through the embankment foundation soil, as shown in Figure 10. The water flow may erode the foundation material, creating a void under the embankment that would weaken the underlying support. Eventually, the embankment would fall into the void and be washed away. The embankment could also fail as a result of downstream slope instability, because the effective stress is very low at the exit face of the flownet associated with the underseepage, which weakens the toe of the downstream slope (Chen and Anderson 1987; Seed et al. 2005; Bonelli 2013).

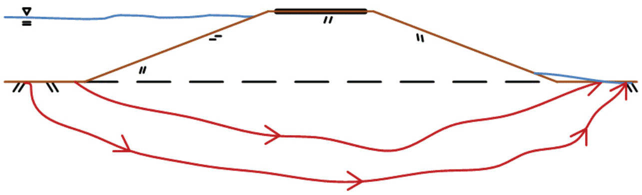

FIGURE 10 Underseepage (after Seed et al. 2005).

The USACE manual *Design and Construction of Levees* presents a number of seepage analysis and mitigation methods. Relevant mitigation structures include cutoffs, riverside blankets, landside seepage berms, pervious toe trench, and pressure relief walls.

WAVE EROSION

Waves produce hydraulic shear stresses which are quite different from those created by unidirectional flood flows. Wave erosion here refers to erosion of the upstream (seaward) slope due to wave action as shown in Figure 11. This type of erosion is different than the wave erosion that would occur on the downstream (landward) side of an embankment, in case the latter was overtopped by water flow or run-up waves.

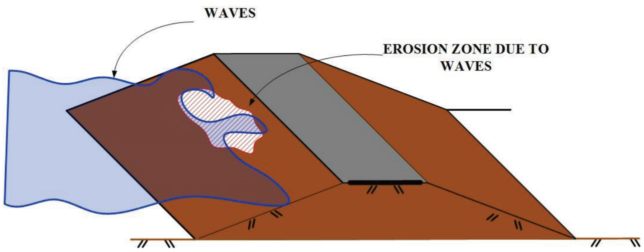

FIGURE 11 Wave erosion (after Seed et al. 2005).

To mitigate the resulting soil erosion and pavement damage, different types of revetments are presented in *HEC-25* (Douglas and Krolak 2008) that could be incorporated into the design.

THROUGH-SEEPAGE AND INTERNAL EROSION

Through-seepage refers to the seepage of water through the embankment. This type of seepage is particularly accentuated if the embankment soil is more pervious than the underlying foundation soil. Internal erosion through the embankment, as shown in Figure 12 (thick lines), may occur when the local hydraulic forces become large enough to wash away particles within the embankment. Such an internal erosion phenomenon can also occur at boundaries and discontinuities between the embankment soil and hard structures such as culverts within the embankment mass. Bonelli (2013) identifies the following mechanisms of erosion associated with seepage through the foundation and embankment.

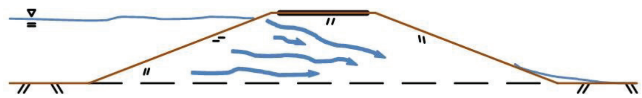

FIGURE 12 Internal erosion (after Seed et al. 2005).

Exit seepage erosion and piping: This mode occurs if the seepage flow becomes sufficient to increase the exit gradient either at the slope face or at the toe. As the flow increases, the hydraulic forces increase; once these forces exceed the erosion resistance of the material, the erosion process begins. The flow net then converges on the whole and further increases the local gradient and the rate of erosion. As a result, the erosion can rapidly "eat back" a tunnel (or "pipe") beneath the levee, hence the name "piping."

Internal seepage erosion: This mode occurs either within the embankment or in the foundation soil. Internal erosion can result from the washing out of finer soil particles as the water flows, which can eventually lead to the collapse of the embankment.

Four phases progressively occur throughout the piping process: (1) initiation of erosion, (2) continuation of erosion, (3) progression to a pipe, and (4) initiation of a breach (Bonelli 2013).

The USACE manual *Design and Construction of Levees* includes such possible mitigation measures as horizontal drainage layers, inclined drainage layers, or toe drains to prevent seepage from emerging on the landside slope. Seepage damage can be mitigated through adequate materials selection and other mitigation measures discussed in chapter four.

LATERAL SLIDING ON FOUNDATIONS

Lateral sliding on foundations occurs when the horizontal water push resulting from rising water equals the lateral resistance of the embankment on its foundation soil. The water pressure on the upstream side has a triangular distribution, which results in a horizontal push. As the water rises on the upstream side, this horizontal push increases and may become equal to the friction force at the interface between the embankment and its foundation soil. At this point, the embankment can start to slide toward the downstream side until the water force decreases to less than the friction force.

Lateral sliding can be prevented if adequate friction is provided at the embankment-foundation interface to resist the imposed hydrostatic lateral forces. Such resistance can be achieved by keying the embankment in the foundation soil to generate a passive resistance situation in the foundation soil mass.

OTHER MODES OF FAILURE: PAVEMENTS AND CULVERTS

Compilation of relevant available information on the failure of the pavement covering the crest of the embankment shows that pavement damage and deterioration can occur in both riverine and coastal embankments due the following mechanisms:

- Undermining of the pavement cover because of back erosion, which generally starts on the downstream side for riverine embankments and on the landward side in coastal embankments.
- Undermining of the pavement cover resulting from wave action, which generally occurs on the seaward side in coastal embankments.
- Undermining due to flow running parallel to the embankment as the storm recedes in coastal environments.
- Rafting, floatation, or highway over-washing as a result of uplift forces created by the penetration of the upstream (seaward) head beneath the pavement.
- Post-flooding pavement deterioration resulting from saturation as explained in the “Softening Due to Saturation” section of this chapter. Further information is available in chapter five (“Pavement Degradation and Failure”).

Based on the survey replies and follow-up interviews, a number of practices have been adopted to minimize pavement damage during flooding. Such practices include selecting rockfill material for the subbase to decrease erosion potential, using an underdrain, keeping the subbase level above the design slope level, and armoring the slopes.

Based on the available information, culverts could cause damage through the following mechanisms:

- Directly, through presenting a weak spot at which erosion initiates (see “Damage in Canyon Environments, Colorado,” in chapter three)
- Indirectly, through being clogged, which leads to overtopping and eventually failure. Examples can be found in the in chapter three section “Overtopping Erosion of a Riverine Highway, Wyoming.”

In general, most DOTs have unique culvert installation methodologies. Relevant literature includes a number of procedures to limit the scour at the inlet and outlet of the culvert, thus delaying erosion. Such guidance can be found in FEMA (2006) and Fell et al. (2008) (discussed in the chapter five section on “Culvert-Related Problems,” and “Culverts” in chapter nine).

SUMMARY

This chapter presented the failure modes in roadway embankments caused by flooding. The chapter presented seven failure modes and possible mitigation measures. The failure modes are overtopping, softening by saturation, underseepage and piping, through-seepage and piping, wave erosion, lateral sliding, and other modes that include culvert and pavement failures. The following chapter presents case examples of these failure modes that were gathered through the survey and follow-up interviews.

CHAPTER THREE

CASE EXAMPLES

INTRODUCTION

This chapter presents 14 case examples gathered from the following six states: Florida, Wyoming, Minnesota, Colorado, Maryland, and West Virginia. To complement the information obtained through the survey, five interviews were conducted with relevant practicing DOT engineers from the aforementioned states (except Florida) who were involved in the case examples. In Florida, a major highway, US-98, suffered severe damage from coastal wave action. In Wyoming, four riverine roadway embankments suffered damage from overtopping. Four overtopping case examples were provided by Minnesota, one of which involved overtopping resulting from wave action. Three cases from arid canyon environments in Colorado are discussed. These cases illustrate the challenges imposed by high-velocity streams, changes in river alignment, and steep embankment slopes. One case is included from Maryland, in which erosion of the embankment riverside slope called for construction of a stone wall to protect the embankment slopes, as explained in "Techniques Used." Lastly, the Kimsey Run project in West Virginia represents a river relocation and stabilization case to protect the embankment from the river. The causes of damage, the protection and stabilization methods selected, and an evaluation of the systems used based on the interviews conducted are presented. A summary of the causes of damage and the selected protection and stabilization measures is included at the end of this chapter.

WAVE EROSION OF A COASTAL HIGHWAY, FLORIDA

The data were provided by documents received from Florida DOT. This case example describes severe damage to a stretch of State Route 30 (US-98) that was caused by Hurricane Dennis. The cause of damage, funding aspect, repair techniques used, and damage from consequent storms are further described here.

Cause of Damage

State Route 30 (US-98) is a major coastal highway in Franklin County, Florida. When Hurricane Dennis struck in 2005, a 14.6-mi stretch of this highway suffered significant damage from wave action and overtopping mechanisms resulting in erosion. Figure 13 shows the damage on the seaward slope. Quick action was taken to armor and protect the stretches of damaged road.

FIGURE 13 Damage from Hurricane Dennis (2005) along US-98, Franklin County, Florida (Courtesy of Florida DOT).

Funding Aspect

Franklin County was declared as one of 13 federal disaster areas in Florida after Hurricane Dennis hit the coast. As a result, the county could use FEMA emergency funds to restore and rebuild important public facilities. The US-98 Carrabelle Revetment project was carried out using FEMA funds.

Techniques Used

The repair work allowed for betterments, including armoring the embankment with sheet piling, soldier piling, articulated concrete blocks, and miscellaneous asphalt and performance turf. Figures 14 through 16 illustrate typical sections of the countermeasures adopted in this project. Figure 17 shows the construction process. Figure 18 shows a completed section of the embankment. The countermeasures were selected with consideration of the right-of-way limitations. Where a slope of 1V to 3H is possible, articulated concrete blocks were installed on the slope above an 18-in.-thick layer of bedding stones placed over filter fabric (Figure 14).

Bermuda sod placed over jute and polymer were used in the finishing soil layer (Figure 15). Figure 16 shows articulated concrete blocks installed down the seaward slope until the edge

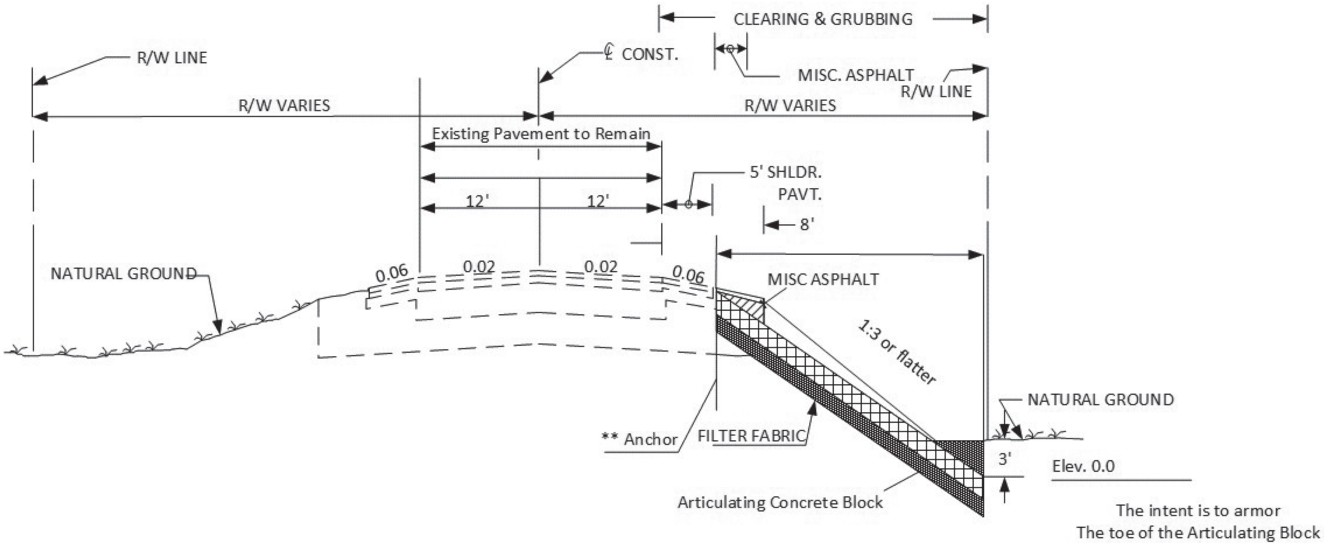

FIGURE 14 Typical armoring detail 1 for 1:3 slopes or flatter (Courtesy of Florida DOT).

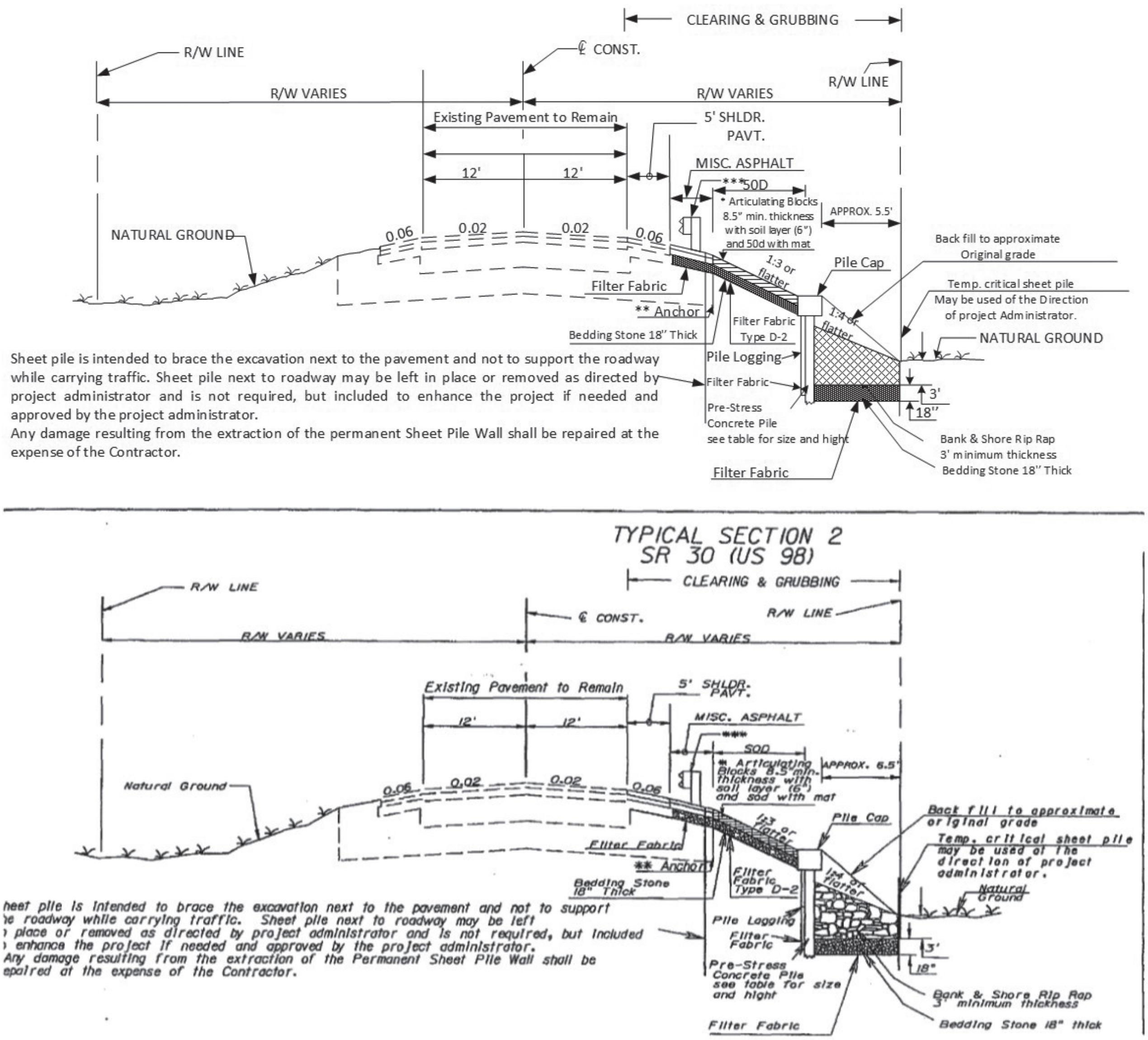

FIGURE 16 Typical armoring detail 2 for slopes steeper than 1:3 (Courtesy of Florida DOT).

of the pile cap. Starting from the other face of the piles (seaward face), riprap was placed over an 18-in. layer of bedding stone that was spread over filter fabric. Figures 17 and 18 show the road under construction and reconstructed, respectively.

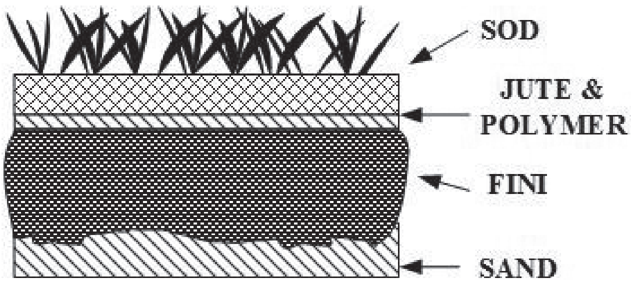

FIGURE 15 Typical vegetation section (Courtesy of Florida DOT).

FIGURE 17 Construction of stabilized roadway section (Courtesy of Florida DOT).


FIGURE 18 Roadway after completion of repair work (Courtesy of Florida DOT).

Damage from Subsequent Storms

Damage from subsequent storms was limited to loss of topsoil and reinforcement turf on the seaward slopes, shown in Figure 19.


FIGURE 19 Damage limited to the removal of performance turf and topsoil after armoring (Courtesy of Florida DOT).

OVERTOPPING EROSION OF A RIVERINE HIGHWAY, WYOMING

The case examples discussed in this section include Childs Draw (Cheyenne), Sand Draw Highway 487, Unnamed Draw Highway 136 (Gas Hill Road near Riverton), and Rock Creek.

Childs Draw, Cheyenne

This newly constructed site is characterized by low fill height. This embankment was designed to sustain overtopping. The downstream slope was protected with gabions (wire-enclosed riprap). The site successfully withstood overtopping and no damage occurred.

Sand Draw Highway 487

A severe flood resulting from rain and snow overtopped the roadway and nearly breached the embankment. Although the exact cause is unknown, overtopping could have occurred as a result of an undersized culvert (it was designed for a 25-year event) or from the culvert being blocked with ice jam. An additional culvert barrel was added to prevent overtopping. No embankment protection was installed. For the past 30 years, the site has not experienced any overtopping.

Unnamed Draw Highway 136 (Gas Hill Road Near Riverton)

This roadway was overtopped during severe flooding likely compounded by an undersized culvert. No embankment protection was installed at the time of the initial design. The downstream slope was repaired by installing gabions. Since then, the site has survived overtopping events without any damage.

Rock Creek

On June 10, 1986, this highway was overtopped because of an upstream dam breach. Overtopping led to damage and the state DOT is now preparing adequate repair measures. Gabions are proposed for this site.

DAMAGE RESULTING FROM OVERTOPPING AND WAVE ACTION OF RIVERINE HIGHWAYS, MINNESOTA

Four case examples from Minnesota are presented herein. TH-220 north of Oslo is an example of wave action inflicting damage on the embankment slopes facing the waves. TH-1 east of Oslo, TH-9 south of Ada, and TH-200 west of Ada are examples of overtopping and associated damage. In general, failure resulting from overtopping is the most prominent roadway embankment problem caused by flooding in the northern Red River Valley. Other parts of Minnesota are subject to summer flash flooding and have steeper terrain and shorter flooding durations. Accordingly, flooding lasted several weeks in the Oslo case examples but only a few days in the Ada cases.

TH-220 North of Oslo

In this case, damage on the downstream slope was caused primarily by wave overtopping. The damage to the unprotected downstream slope is shown in Figure 20. In this case, paving the downstream slope of the embankment minimized the damage caused by wave action. Figure 21 shows the actual paving of the embankment slope, and Figure 22 shows the minimal damage sustained during a subsequent flood after the adoption of the slope paving solution.

FIGURE 20 Wave damage on the downstream unprotected slope (TH-220 north of Oslo, Courtesy of Minnesota DOT).

Issues related to the workability of paving slopes can limit the use of this technique in certain cases. Improper handling can affect the asphalt's durability. Segregation and variation in density as a result of improper compaction can lead to faster crack development, which would lead to an increase in uplift potential. Nevertheless, this option is cost-effective in areas subject to slow or stagnant currents. To increase paving slopes' durability, proper placement and adequate maintenance are important. Maintenance would include weed spraying and fog sealing (Lim and Anderson 2011). It is important to pave far down the slope, practically to the expected tailwater level.

FIGURE 21 Placing pavement on the slope (TH-220 north of Oslo, Courtesy of Minnesota DOT).

FIGURE 22 Wave damage after placing pavement on the downstream slope (TH-220 north of Oslo, Courtesy of Minnesota DOT).

TH-1 East of Oslo

During the 2011 spring flooding events, overtopping of this roadway embankment lasted for weeks. At the time of the overtopping, the downstream slopes were armored with a closed-cell articulate concrete block system. The system succeeded in resisting the long-term overtopping. Prior to placing the articulated concrete block system, a coarse gradation (OGAP) system was attempted but it failed in a previous 2011 flooding event. The two systems are discussed herein.

Articulate Concrete Block System

The system consists of blocks that are placed over a geotextile blanket. Pavement or topsoil and turf are then placed over the blocks. The system successfully withstood the 2011 flooding forces. No damage to the downstream slope or the pavement occurred. As shown in Figure 23, the newly installed topsoil was lost, and no turf was established at the time of the flood. Now that the turf is fully developed, it will likely perform better in future flooding events.

FIGURE 23 Loss of topsoil in the 2011 event after the installation of articulated concrete blocks on the downstream slope (TH-1 east of Oslo, Courtesy of Minnesota DOT).

The open-cell articulated concrete blocks were initially recommended from a design point of view. Yet the contractor included pyramid solid/closed blocks in his bid, and eventually pyramid solid blocks were used. However, the spacing between the pyramid blocks did meet the criterion for open-cell system.

Systems Prior to Articulate Concrete Blocks

The downstream slope of TH-1 was initially designed as shown in the typical sections of Figures 24 and 25. Course gradation (OGAB) placed on top of a geotextile layer was used to enhance the resistance of the downstream slopes against erosive forces. Yet the hydraulic forces exerted during the flood far exceeded the granular materials' erosion resistance, and the system failed (Figure 26).

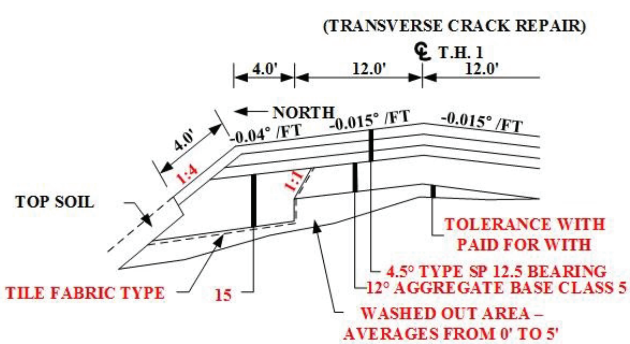

FIGURE 24 Failing typical Section 1 in 2010 flood (Courtesy of Minnesota DOT).

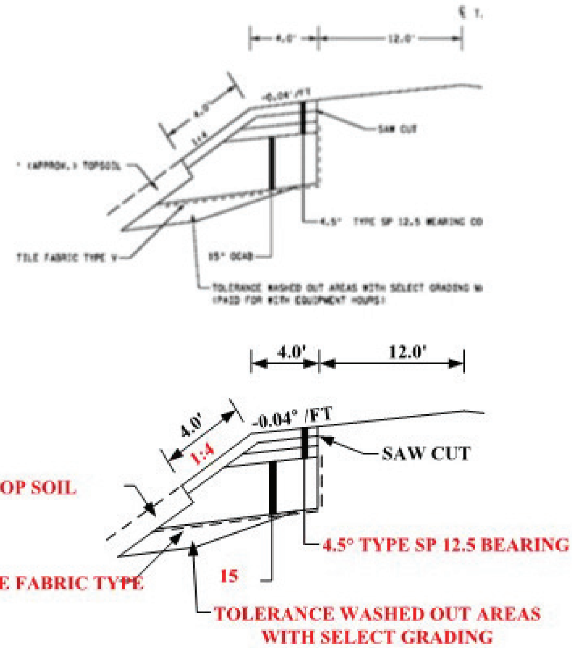

FIGURE 25 Failing typical Section 2 in 2010 flood (Courtesy of Minnesota DOT).

FIGURE 26 Overtopping damage in 2009 flood (TH-1 east of Oslo, Courtesy of Minnesota DOT).

TH-9 South of Ada

In 2009, the TH-9 embankment was overtopped for a few days during flooding. This embankment is characterized by relatively flatter slopes; as a result, the velocities associated with overtopping were relatively low. Figure 27 shows the damage inflicted on the downstream slope. Riprap was adopted as a protection measure and placed on the downstream slope of the embankment (Figure 28). The riprap was then buried under an asphalt layer owing to such safety concerns as cars and snowmobiles accidentally running off the roads and hitting the riprap. The embankment resisted the 2011 flooding without any damage to the downstream slope or the pavement, as shown in Figure 28.

FIGURE 27 Overtopping damage in 2009 flood (TH-9 south of Ada, Courtesy of Minnesota DOT).

FIGURE 28 Installation of riprap placed over geotextile and then paved over (TH-9 south of Ada, Courtesy of Minnesota DOT).

FIGURE 29 Successful performance of installed riprap system and pavement placed over it post-2011 flood (TH-9 south of Ada, Courtesy of Minnesota DOT).

Also shown in Figure 28, riprap was used to protect the downstream slope against anticipated overtopping. The riprap was then buried under an asphalt layer owing to safety concerns. Such concerns include cars accidentally running off the roads and snowmobiles. The embankment endured the 2011 flooding forces without any damage to the downstream slope or the pavement as revealed in Figure 29.

TH-200 West of Ada

This case presents a failure of the pavement applied to an embankment's downstream slope to protect against overtopping. The 2009 flood caused partial erosion and pavement damage on the downstream slope. The asphalt pavement was used on the downstream slope and failed, as shown in Figure 30. Thus, this technique appears to perform well to protect against wave overtopping in rivers, but not for prolonged overtopping in which higher velocities would occur.

FIGURE 30 Failed downstream slopes with asphalt pavement placed on them in 2009 flood (TH-200 west of Ada, Courtesy of Minnesota DOT).

Articulated concrete blocks were then constructed over the downstream slope. The system's performance, however, has not yet been tested in this location because overtopping has not occurred since 2010.

Practical and Financial Considerations

Paving a slope is more difficult than paving a flat surface. Construction challenges involve aggregate segregation and difficulties compacting the asphalt on a sloping surface. Such difficulties affect durability; therefore, sloped pavements tend to deteriorate and crack faster. As a result, they may perform poorly during flooding events because water may infiltrate the cracks and contribute to lifting the pavement. In this case, erosion protection will be limited. The case example implies that paved slopes are more effective in protecting against wave action than against overtopping erosion.

In terms of cost, grass is the least expensive material option, but it is limited to relatively low flow velocities and associated low hydraulic shear stresses. Grass is effective as long as the applied hydraulic shear stress does not exceed the critical shear strength of the grass cover; otherwise, the cover will fail and the underlying soil will be progressively eroded.

Pavement is the next least costly solution. Articulated blocks and riprap (with underlying geosynthetic fabric) are good means of preventing overtopping damage, but they are more expensive means of protecting against overtopping than pavement. Table 3 shows prices for riprap and articulate concrete blocks based on two projects in Minnesota.

TABLE 3
PRICES FOR RIPRAP AND ARTICULATE BLOCK SYSTEMS IN MINNESOTA

Protection System	Project Year	In-place Price ($ per sq. yard)	Comments
Riprap	2012	27.10	Class 3, 12” nominal, 18” max, including geotextile fabric
Articulated Block Mat	2011	57.65	Open cell type 2 including geotextile fabric

Maintenance Considerations

In flood-prone areas, efforts are made to seal thermal and reflective cracks before flooding events. If left unsealed, water can infiltrate the cracks and cause internal erosion. Once flooding occurs, limited access makes sealing the cracks difficult.

Another consideration is the impact of saturation on pavement strength. The potential for embankment saturation is generally considered when the water height on the upstream slope reaches two-thirds the height of the embankment. The long-term effect of embankment saturation on the pavement placed over the embankment has not been directly observed, but the following example gives a clue. The northern Red River Valley area is predominantly agricultural. Spring planting creates a flurry of heavy equipment and trucks on roadway embankments that may not yet be fully recovered from saturation caused by high water in the spring. Some premature pavement deterioration has been observed as a result of increased heavy trucks and equipment.

DAMAGE IN CANYON ENVIRONMENTS, COLORADO

These three Colorado case examples lie in an arid canyon environment. In each case, the damage was caused by the intense prolonged rain and high-velocity runoffs caused by the September 2013 storm event. The embankments in these cases are generally characterized by steep slopes (2:1 or 1:1 slopes). The slopes were generally unprotected, but the river changed its course abruptly during the storm and eroded the roadway embankment.

SH-7 MP 26.90–27.68

In this case, the Middle St. Vrain Creek changed its course, which led to damage along SH-7. The high-velocity runoff (with debris) eroded the embankment toe. Eventually, stretches of the embankment were completely eroded. The damage was located at the outer arc of the meandering bends. The failures at measuring points (MPs) 26.90, 27.68, and 27.28 are shown in Figures 31, 32, and 33, respectively.

FIGURE 31 SH-7 damage at MP 26.90 (Courtesy of Colorado DOT).

Emergency repair work was performed, which included the placement of rock-fill embankments (considered more resistant to erosion) and riprap on the embankment slopes in the vicinity of the creek at specified locations.

US-34 MP 75.05–76.73

The damage occurred along the stretch of US-34 that runs next to Big Thompson River. The high-velocity flows eroded the embankment at the outer curve of meandering bends in the vicinity of the embankment toe. The damage was aggravated at certain locations because of the presence of drainage elements. These elements included highway culverts under the roadways that drain mountainside runoff into the river and private access bridges (private culverts) that connect the highway to the properties on the other side of the river. Along this stretch of highway, the access bridges were severely undersized for such extreme flow rates and created intense backwater. This backwater resulted in turbulent flows that washed out the embankment and completely destroyed the roadway as shown in Figures 34, 35, and 36 at MPs 75.05, 76.40, and 76.73, respectively, in the vicinity of private culverts along US-34.

FIGURE 32 SH-7 damage at MP 27.68 (Courtesy of Colorado DOT).

FIGURE 33 SH-7 damage at MP 27.28 (Courtesy of Colorado DOT).

FIGURE 34 US-34 damage at MP 75.05 (Courtesy of Colorado DOT).

FIGURE 35 US-34 damage at MP 76.40 (Courtesy of Colorado DOT).

Temporary repair work included placing rockfill and riprap at certain locations. For the permanent repair design, the impact of the access bridge locations on the overall performance of the flow and roadway embankment are currently being assessed. It is undecided whether the culvert bridges will be rebuilt with special provisions or protections to minimize damage, or whether in the event of flooding the culvert bridges will be considered sacrificial structures, to maintain roadway embankment stability.

FIGURE 36 US-34 damage at MP 76.73 (Courtesy of Colorado DOT).

US-36: 7.70–8.00

In this case, the damage occurred along a stretch of highway that had been subject to realignment. The narrow channel of the Little Thompson River caused high-velocity flows at MP 7.90–8.00 and the erosion proceeded laterally as the water level rose.

The riverbed was essentially at the level of the bedrock, but the steep embankment slopes failed as a result of toe erosion (coupled with softening and the corresponding loss of strength from the extreme amounts of rain). Portions of the road were removed by the flow. A hydraulic jump occurred downstream as the water plunged straight downwards and caused severe erosion.

Two factors are likely to have contributed to this erosion damage: (1) the realignment of a river when rivers typically tend to meander, and (2) the area's geology (rock riverbed versus relatively finer embankment material).

At MP 7.70, one lane of the road was eroded most likely because the embankment material in this area was more erodible (weak point). This led to the structural failure of the overlying pavement (Figure 37). At MP 7.80, the roadway shoulder was lost (Figure 38), probably from slope failure resulting from toe erosion coupled with embankment softening during heavy rain.

At MP 7.90, downstream from MP 7.80, several feet of the embankment shoulder were lost. At MP 8.00, a 60-ft drop in the riverbed developed owing to the hydraulic jump that formed at that location. The water plunged from the bedrock bottom into the relatively softer material (Figure 39).

FIGURE 37 US-36 damage at MP 7.70 (Courtesy of Colorado DOT).

FIGURE 38 US-36 damage at MP 7.80 (Courtesy of Colorado DOT).

FIGURE 39 US-36 damage at MP 8.00 (Courtesy of Colorado DOT).

Funding Agency

The repair work was funded by FHWA Emergency Relief (ER). One of the key features to achieving the repair work within the assigned time and budget was the constructive collaboration between the parties involved.

When disasters strike, it is not a straightforward process to allocate the funds and identify the responsible agencies while preparing for the temporary design. This process went relatively smoothly as a result of all the projected efforts of the teams involved. FHWA issued a quick release of the funds so that the repair work could be started right away. Within about 10 days, the contractors started work on a number of sites.

Design Methodology

The design approach included one-dimensional (1-D) and two-dimensional (2-D) hydraulic modeling to identify the potential high-risk sources. The actual peak flows in the 2013 flood were estimated at several locations, and a hydrological study was carried out. The revised 100-year floods were calculated accordingly and will be used in the permanent design.

Discussion of Potential Protection Systems

FHWA ER funds were allocated for emergency repair of the damage caused by the September 2013 storm event. Based on FHWA funding regulations, two phases of design and construction are carried out: the emergency repair and the permanent repair. The emergency repair phase restores the highway to its initial condition, which restores normal traffic flow and ensures that travelers can reach their destinations. The adopted solutions for this stage were rockfill embankments and riprap protection for certain areas.

The permanent repair (currently under preparation) considered the different viable solutions throughout the project's lifetime. The relevant design would elaborate on the type and extent of protection adequate to mitigate the risks faced by the highway. In general, minimizing damage to roadway embankments from flooding is approached through either channel restoration to enhance the structure's performance, or embankment alteration. If overtopping is foreseen, rock-fill can be used as embankment material because it is more resistant to erosion and easier to place. The following embankment protection options are discussed further: vegetation, gabions, riprap, paved slopes, articulated concrete blocks, and walls.

Because of Colorado's arid climate, the use of vegetation is limited in some areas. The arid climate has also led Colorado DOT away from using gabions, which is now noted in the state provisions. *HEC-14* recommends against using gabions in arid climate owing to the rupture of the wires of the baskets. Riprap is used to protect some slopes. The use of geotextile under riprap, however, is dependent on the type of repair (emergency versus permanent). Geotextile is generally used under riprap for permanent repairs, but not for temporary repairs. Paved slopes are generally not used, due to the arid climate that causes cracks in the pavement placed on the slopes. Articulated concrete blocks, on the other hand, could be an option especially because such systems can be applied on steep slopes (2:1 or 1:1). Also, as the slopes get steeper, building walls remains an option. Wall construction to protect roadway slopes has been used since 1976. Several walls were placed on rocks (found at shallow depths) at the base of the canyons. During the floods, the walls withstood the water forces and limited erosion. The use of walls does present two relevant concerns: scour at the bottom of the wall and erosion around or in the vicinity of the wall can occur, and, depending on the flood discharge, water might get behind the wall. When the bedrock is shallow, walls can be easily used; however, for sites in which rock cannot be found even at 40-ft depths, walls are more difficult to use.

MD-24 DEER CREEK STREAM STABILIZATION, MARYLAND

This section presents the stabilization of MD-24 Deer Creek Stream in Maryland based on the information obtained through the interview. The cause of damage, site characteristics, stabilization methods, and typical sections are included herein.

Cause of Damage

The roadway embankment runs parallel to the stream channel. When the stream water level rose, the unprotected embankment slope was eroded. The damage to the embankment slope accumulated from a number of flood events, which required a project to stabilize the embankment running along the streamside. This project was funded by the state and by FHWA.

Techniques Used

An implicated stone wall (Figures 40 and 41) solution was adopted to protect the roadway embankment against erosion. The stones were selected based on a number of considerations to satisfy the requirements set by several parties including the community panel established for the purpose of this project and relevant environmental and federal regulations.

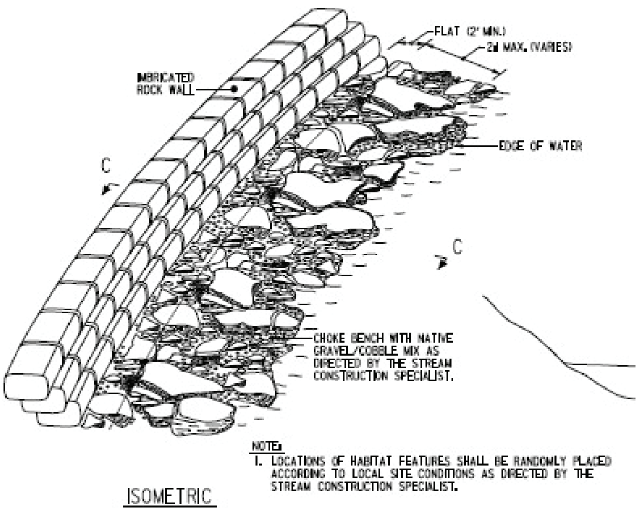

FIGURE 40 Implicated stone wall schematic drawing (Courtesy of Maryland DOT).

FIGURE 41 Implicated stone wall under construction (Courtesy of Maryland DOT).

The cross sections varied depending on the characteristics of the subsurface bearing stratigraphy (soil/rock) and the available distance to the designed stone wall. Figure 42 shows a typical retaining wall section and Figure 43 presents a reinforced wall section.

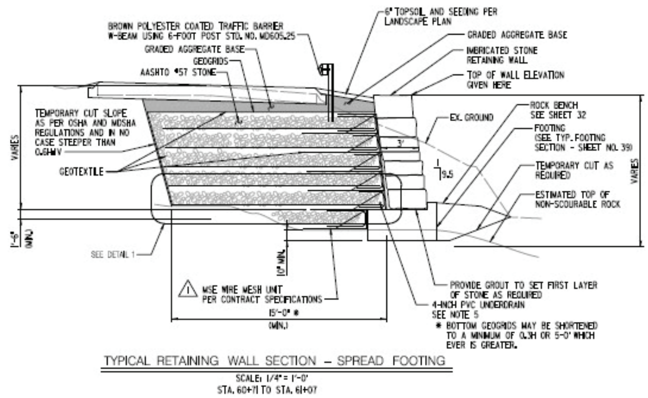

FIGURE 42 Typical retaining wall section—spread footing (Courtesy of Maryland DOT).

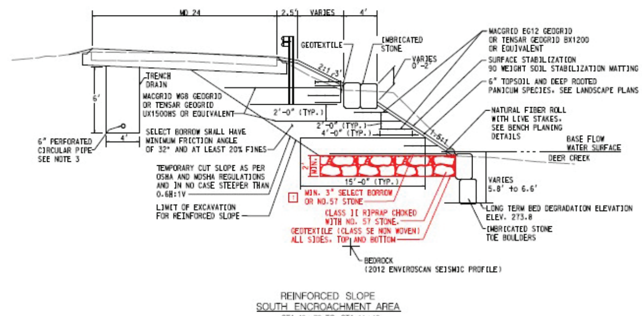

FIGURE 43 Reinforced slope section (Courtesy of Maryland DOT).

KIMSEY RUN PROJECT, WEST VIRGINIA

The Kimsey Run Project presents a case in which a stream jumped out of its course during a flooding event and overtopped a nearby embankment. Two flooding events caused embankment damage; thus, repair work was carried out twice: after Hurricane Isabel in 2003 and about 10 years later. After the first flooding event, the stream was shifted back and stabilized. After the second flood, the river channel was reconstructed. In both cases, the repair work was carried out using state funds. The causes of damage and the techniques used in the stream stabilization are presented herein.

Cause of Damage

The roadway embankment initially lay in the vicinity of a river meander. During Hurricane Isabel in 2003, the river jumped out of its initial course at the location of the meander bend. As a result, the neighboring embankment was overtopped.

The resulting damage included erosion of the embankment slope facing the stream as the water rose to the embankment toe (Figure 44) and pavement rafting (Figure 45). Little damage was inflicted on the other embankment slope, which was much flatter.

FIGURE 44 Slope erosion and pavement undermining after Hurricane Isabel in 2003 (Courtesy of WVDOT).

FIGURE 45 Pavement rafting after Hurricane Isabel in 2003 (Courtesy of WVDOT).

Although not verified, another factor that could have affected the stream stability was the pond that was built just downstream of the site location. This pond was built by NRCS (National Resources Conservation Service) to reduce the flooding discharge downstream.

Techniques Used After the First Flood

After Hurricane Isabel in 2003, surveying work was first carried out to map the area and its components. The river channel was relocated and reconstructed further away from the roadway embankment. In-stream structures, namely cross vanes and rock vanes, were used in the channel to divert the water away from the embankment and to decrease the stream velocities at the bend location. Additionally, hardwood trees that were lost during the storms were used along the channel to strengthen the channel bank. A photograph of the completed system is shown in Figure 46. This protection system functioned well for about 9 years. After that, the stream got behind the established bank protection and started to erode the bank slopes near the roadway embankment (Figure 47). This led to further reconstruction of the channel.

FIGURE 46 Channel reconstruction after Hurricane Isabel in 2003 (Courtesy of WVDOT).

FIGURE 47 Erosion at the bend location (Courtesy of WVDOT).

Based on this project, in-stream structures worked for a while, but they were not a sustainable solution. The rocks were embedded deep enough in the stream bed to remain stable. But as the water level rose, the water would go over and around the structures and eventually reach the embankments. A possible solution might have been to further increase the length of the rock veins across the flood area.

The use of the tree trunks, which were supposed to strengthen the bank, resulted in more erosion around those obstructions. The effectiveness of this technique might be related to how deep the trunks are embedded in the bank. Based on this project, this solution would need further development since, instead of strengthening the stream bank, it caused more local erosion.

Techniques Used After the Second Flood

The second repair work included armoring the bank with larger riprap, as shown in Figure 48. No geotextile was used for this purpose. The underlying materials were mostly cobbles that were considered good filter material. So far, the riprap has worked, but is not necessarily ultimately effective. In the future, protection of the embankment slopes would probably be considered as an option.

FIGURE 48 Repairing bank with riprap (Courtesy of WVDOT).

SUMMARY

This chapter presented several case examples. It summarized the causes of damage and the techniques used (Table 4). The causes of damage varied among these cases, as did the protection techniques used. The direct causes of damage included wave action, overtopping, overtopping from waves, and channel migration. The repair techniques included articulated concrete blocks, gabions, riprap, stone walls, in-stream structures, vegetation, paving of slopes, and relocation of streams. Based on these case examples, the success of the protection measures is found to be site and case specific.

TABLE 4
SUMMARY OF THE CASE STUDIES

Case Study	Description	Repair Techniques Used	Evaluation of Techniques
State Route 30 (US-98), Florida	Significant damage due to coastal wave action	Seaward slope protected using sheet piling, soldier piles, articulate concrete blocks, and assorted asphalt and performance turf	Damage from subsequent storm was limited to loss of topsoil and turf reinforcement
Childs Draw, Cheyenne, Wyoming	Embankment designed for overtopping	Downstream slope protected with gabions	No damage
Sand Draw Highway 48, Wyoming	Overtopping due to undersized culvert or blockage of culvert with ice jam	Additional culvert installed	Site has not experienced overtopping in 30 years
Unnamed Draw Highway 136, Wyoming	Overtopping due to severe flooding or undersized culvert	Downstream slope protected with gabions	No damage
Rock Creek, Wyoming	Overtopping due to dam breach	Gabions proposed to protect downstream slope	_
TH-220 North of Oslo, Minnesota	Overtopping by waves	Downstream slope paved	Minimal damage
TH-1 East of Oslo, Minnesota	Overtopping by floods in 2009 (Trial 1) and 2011 (Trial 2)	Trial 1: Employed coarse gradation on downslope (OGAP) Trial 2: Used articulate concrete blocks	Trial 1: Failure Trial 2: Loss of newly installed topsoil
TH-9 South of Ada, Minnesota	Overtopping	Paved-over riprap placed on the downstream slope	No damage
TH-200 West of Ada, Minnesota	Overtopping	Trial 1: Paved downstream slope Trial 2: Used articulate concrete blocks	Trial 1: Damage to downstream slope Trial 2: No overtopping yet
SH-7 MP 26.90-27.68, Colorado US-34 MP 75.05-76.7, Colorado US-36: 7.70-8.00, Colorado	Severe erosion of the embankment riverside slope Aggravated erosion at relatively weaker locations and at culvert locations; slope failures due to saturation	Emergency repair: Rockfill embankments and riprap placed on selected locations on the riverside slope	Under study
MD-24 Deer Creek Stream Stabilization, Maryland	Accumulated erosion of the embankment riverside slope from a number of flooding events	Implicated stone wall coupled with reinforced slopes installed	Under construction
Kimsey Run Project, West Virginia	Embankment damage due to stream changing its course in two flooding events	Trial 1: Bank relocated and reconstructed using in-stream structures and tree trunks Trial 2: Bank armored with riprap	Trial 1: Lasted for about 10 years Trial 2: No damage so far

CHAPTER FOUR

HYDROLOGIC AND HYDRAULIC FACTORS

INTRODUCTION

Hydrologic and hydraulic factors are extremely important in the design of embankments subjected to flooding. These factors control the volume of water that is likely to flood the embankment's surroundings, the duration of the flood, and the water surface elevation. These factors affect the design against nearly all failure modes and in particular the overtopping failure mode and the seepage failure mode. However, the selection and estimation of these factors is not a straightforward process. It is fundamental to couple every decision with engineering judgment to consider, to a feasible extent, site variability and constraints.

The goal of this chapter is to present hydrologic and hydraulic concepts that could be employed in the design of roadway embankments subjected to flooding. The information included herein is based on available literature and guided by the survey responses. The following useful concepts shed light on some important design considerations.

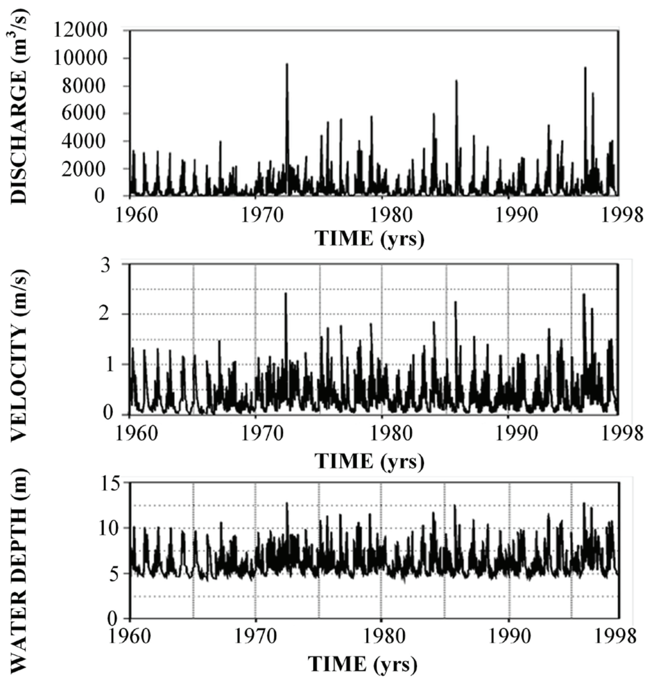

FIGURE 49 Hydrographs representing daily peak flow discharge, velocity, and water depth (Briaud 2013).

USEFUL CONCEPTS

This section presents useful concepts on the selection of the design flood frequency and the impact of the flood frequency selection on such design parameters as discharge and velocity. A brief description of coastal parameters is also included.

Design Flood Frequency

One of the most important hydrologic factors to be considered in the design of embankments subjected to flooding is the design flood. This design flood is chosen on the basis of the recurrence interval, also called the return period. The 1% chance flood (100-year flood) and the associated river flow discharge Q_{100} are often used. The 100-year flow discharge Q_{100} (m^3/s) is the discharge that has a 1/100 probability of being exceeded in any one year. Another commonly considered flood is the 500-year flood and the 500-year river flow discharge Q_{500} (m^3/s), which has a 1/500 probability of being exceeded in any one year. One simple way to obtain the flow discharge value for these floods is to collect the flow history as a function of time (flow hydrograph, Figure 49). The flow discharge is typically collected through a stream gage placed along a river or regression of stream gage data for ungaged sites.

The following simple graphical method serves as a good illustration of the process. This method (e.g., Chow et al. 1988) consists of obtaining the yearly maximum flow parameters from the hydrograph, ranking them in descending order of intensity, calculating for each flow the probability of exceedance as the rank divided by the total number of observations + 1, then plotting the flow versus the probability of exceedance on a semi log paper such as the one in Figure 50. Once the data are plotted, a linear regression is performed over, say, the first 20 to 30 years of data and extrapolated to the 0.01 probability of exceedance for the 100-year flood and to the 0.002 probability of exceedance for the 500-year flood. The return period is the inverse of the probability of exceedance. There are other and more refined ways of obtaining these design floods, but this simple graphical method helps one to understand the process and the meaning of a 100-year flood: a flood that has a 1% chance of being exceeded in any one year. Figure 50 shows the result of an analysis for the hydrograph at the Woodrow Wilson Bridge. As that figure illustrates, the 100-year flood has a flow discharge of 12,600 m^3/s and the 500-year flood has a value of 16,600 m^3/s.

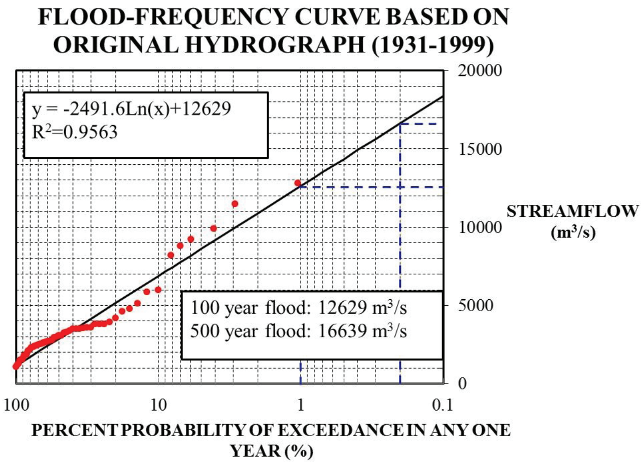

FIGURE 50 Flood frequency curve obtained from a measured flow discharge hydrograph (Briaud 2013).

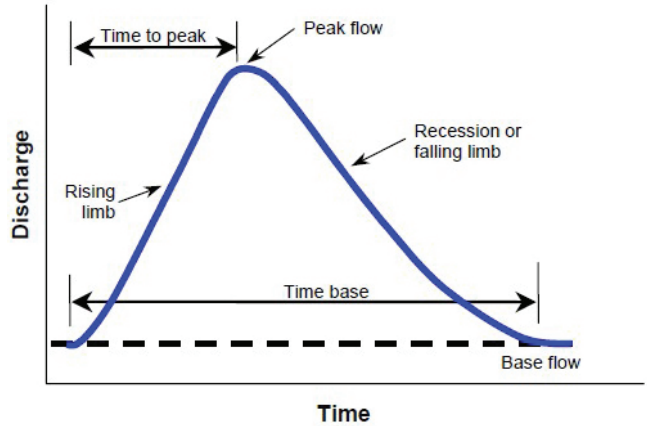

FIGURE 51 Elements of a flood hydrograph (*Source*: HDS2).

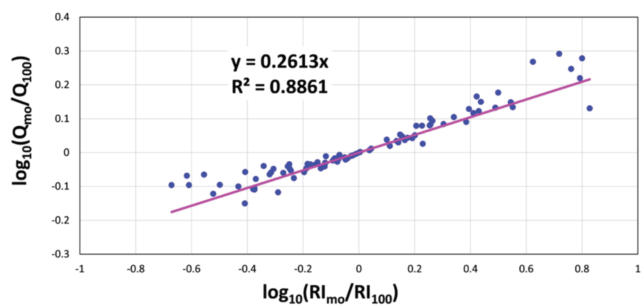

FIGURE 52 Relationship between flow discharge Q and recurrence interval RI (Briaud et al. 2009).

Note that the example in Figure 49 shows a hydrograph for a period of nearly 40 years. However, the hydrograph could be for a much shorter period of time, such as one single flood where the hydrograph may be only 48 hours long and show the increase and decrease of velocity during the flood (Figure 51). This might be called a single storm or flood hydrograph. A storm hydrograph shows how a drainage basin responds to rainfall during a storm event. This type of hydrograph covers a relatively short period, for instance, hours or days. It is useful in the applications that require knowledge of the flow discharge over time. The peak flow discharge is of interest to the designers because the structures are sized accordingly, but the hydrograph provides a higher level of precision.

Hydrographs provide useful information about the anticipated flood, such as the duration of overtopping. Many methods are available for the computation of hydrographs, and can be found in such references as HDS-2 (McCuen et al. 2002). When sufficient data are available, the computations can be done without difficulties. However, if such data are not available, synthetic methods can be used. The selection and reliability of an adequate method in this case requires a clear understanding of the site conditions and of the existing methods coupled with engineering judgment.

An example of a storm hydrograph is shown in Figure 51. Its main elements are the base flow, the time base, the time to peak, the rising and recession limbs, and the peak flow discharge.

Flow Discharge Versus Recurrence Interval

Note that there is a nonlinear relationship between the flow discharge and the recurrence interval or return period. The following model was found to fit the data well for river flow gages in Texas (Briaud et al. 2009; Figure 52).

$$\left(\frac{Q}{Q_{100}}\right) = \left(\frac{RI}{RI_{100}}\right)^{0.261} \tag{4.1}$$

Where:

– Q is the flow discharge,

– Q_{100} is the flow discharge for the 100-year flood,

– RI is the recurrence interval also known as the return period, and

– and RI_{100} is the 100-year recurrence interval equal to 100.

Equation 4.1 indicates, for example, that for the 500-year flood, the RI ratio is 5 but the flow discharge ratio is 1.52. In other words, the RI is 500% larger but the flow discharge is only 52% larger.

Velocity and Water Depth Versus Flow Discharge

Once the design flood is selected and the corresponding flow discharge Q is known, it is very useful to obtain the velocity and the water depth that corresponds to each flow value. Let's start with the relationship between the flow discharge and the velocity. Using Manning's equation and the definition of the hydraulic radius, the following relationship for a rectangular channel cross section appears reasonable (Briaud et al. 2009):

$$\left(\frac{v}{v_{100}}\right)=\left(\frac{Q}{Q_{100}}\right)^{0.4} \tag{4.2}$$

Where:

– v is the velocity,

– v_{100} is the velocity for the 100-year flood,

– Q is the discharge corresponding to v, and

– Q_{100} is the discharge for the 100-year flood.

For a triangular channel cross section, the same approach gives the following:

$$\left(\frac{v}{v_{100}}\right)=\left(\frac{Q}{Q_{100}}\right)^{0.25} \tag{4.3}$$

Considering that natural river channels are closer to being rectangular (exponent = 0.4) than triangular (exponent = 0.25), an average exponent of 0.35 is selected and the relationship is

$$\left(\frac{v}{v_{100}}\right)=\left(\frac{Q}{Q_{100}}\right)^{0.35} \tag{4.4}$$

Similarly, the relationship between the water depth and the flow discharge can be found by using Manning's equation and the definition of the hydraulic radius. For a rectangular channel cross section, Briaud et al. (2009) give:

$$\left(\frac{y}{y_{100}}\right)=\left(\frac{Q}{Q_{100}}\right)^{0.6} \tag{4.5}$$

Where:

– y is the water depth,

– y_{100} is the water depth for the 100-year flood,

– Q is the discharge corresponding to y, and

– Q_{100} is the discharge for the 100-year flood.

For a triangular channel cross section, the same approach gives

$$\left(\frac{y}{y_{100}}\right)=\left(\frac{Q}{Q_{100}}\right)^{0.375} \tag{4.6}$$

Considering that natural river channels are closer to being rectangular (exponent = 0.6) than triangular (exponent = 0.375), an average exponent of 0.525 is selected and the relationship is

$$\left(\frac{y}{y_{100}}\right)=\left(\frac{Q}{Q_{100}}\right)^{0.525} \tag{4.7}$$

Impact of Recurrence Interval on Velocity and Water Depth

Equations 4.1–4.7 can be combined to link the velocity and the water depth to the recurrence interval as follows:

$$\left(\frac{v}{v_{100}}\right)=\left(\frac{Q}{Q_{100}}\right)^{0.35}=\left(\left(\frac{RI}{RI_{100}}\right)^{0.261}\right)^{0.35}=\left(\frac{RI}{RI_{100}}\right)^{0.091} \tag{4.8}$$

$$\left(\frac{y}{y_{100}}\right)=\left(\frac{Q}{Q_{100}}\right)^{0.525}=\left(\left(\frac{RI}{RI_{100}}\right)^{0.261}\right)^{0.525}=\left(\frac{RI}{RI_{100}}\right)^{0.137} \tag{4.9}$$

Where:

– v is the velocity,

– y is the water depth,

– Q is the discharge,

– RI is the recurrence interval,

– v_{100} is the velocity for the 100-year flood,

– y_{100} is the water depth for the 100-year flood,

– Q_{100} is the discharge for the 100-year flood, and

– RI_{100} is the 100-year recurrence interval.

Equations 4.8 and 4.9 convey a very important message: the velocity and the water depth are not very sensitive to the recurrence interval. For example, if the *RI* ratio is 5, as in the case of the ratio between the 500-year flood and the 100-year flood, the velocity ratio is 1.16. In other words, when the *RI* is 500% larger, the velocity is only 16% larger. For the same example, the water depth ratio is 1.25 or the velocity is 25% larger. As another example, if one were to apply a factor of safety equal to 1.5 on the 100-year flood velocity, it would be equivalent to using the 10,000-year flood as an extreme event. This is what is done in the Netherlands. If instead the 500-year flood is considered as the extreme event, the factor of safety on the velocity is 1.16.

Coastal Parameters

The parameters to be considered in a coastal embankment subject to hydrodynamic forces depend on the nature of the flow. Two failure modes that cause surficial erosion were identified in chapter two: overtopping failure mode with three flow mechanisms ("Overtopping" in chapter two) and wave erosion ("Wave Erosion" in chapter two). The main parameters required for coastal analysis and design are namely the height of surge above the roadway crest for overtopping: **Case I**, the design run-up wave elevation for **Case II**, and the design significant wave height for **Case III**. To design for wave action on the seaward slope of the embankment, the design wave height and wave run-up elevation are required. This will be further explained in this chapter.

The selection of a design wave, a design water level, or a recurrence interval is, as in the case of river embankments, a very critical issue. Generally, recurrence intervals between 25 years and 50 years (4% and 2% yearly probability of exceedance) are adopted in coastal projects (Douglas and Krolak 2008; FHWA 2014). The recurrence interval affects the design wave height and, therefore, the riprap stone size and the extent of armoring (Douglas and Krolak 2008). As a result, the adoption of a risk-based approach in the selection of a "design recurrence interval" with consideration given to the design life of the structure is important. This will be further discussed in chapter seven.

HYDROLOGICAL METHODS AND CONSIDERATIONS

Riverine Hydrology

Estimating peak discharges for various recurrence intervals is a challenging task. Selecting an adequate method depends on whether the site is gaged or ungaged. Gaged sites are located at or in the vicinity of a gaging station. If continuous streamflow records exist for a sufficient period of time, statistical methods are applied to determine the peak discharge. Ungaged sites are sites that are not at or near a gaging station and do not have relevant streamflow record. For this purpose, deterministic methods are available. The accuracy of these methods is dependent on the availability of relevant data. Because data stations are generally unevenly distributed, the calculated values are only estimations of the probable values. The methods currently used based on the survey results are described here.

Hydrological Means and Methods

If available for the site, gage data are preferred for the basis of estimating peak discharges. The key issue is to have sufficient recorded data for a continuous period of time. Otherwise, existing studies (including flooding case examples, FEMA studies, and regression equations) can be used as well as rainfall-runoff models.

Gaged Sites

Based on current DOT practice, a number of methods are used to obtain the peak flows in a river. The designer selects the most appropriate method while considering the limitations of each method coupled with engineering judgment.

The methods used to obtain the peak flow in a river are listed from the most commonly used to the least commonly used:

- Direct Available Data can be obtained through the U.S. Geological Survey (USGS) Streamflow Information Program. This database includes gage data for gaged sites and is available online (http://water.usgs.gov/nsip/). Other sources of information are listed in the Federal Lands Highway manual (FHWA 2014). This manual includes, in addition to gage data, such state-specific studies as peak-flow frequency estimates for gaged sites and magnitudes of flood flows for selected annual exceedance probabilities. When statistical analysis of raw data are used, the guidelines in Bulletin 17B (USGS 1982) are followed; in particular, the log Pearson Type III distribution is adopted.
- Regional regression equations have been developed for almost all states to estimate peak-streamflow frequency for ungaged sites in natural basins. This involves using peak streamflow frequency data from gaging stations in natural basins. For some areas, urban regression equations have been developed and are being used.
- FEMA Flood Insurance Studies are often available where highway facilities encroach upon established or planned regulatory floodplains. A flood frequency curve approved by FEMA for the site may be available. These studies may indicate if an embankment is subject to flooding and the likely recurrence interval of the peak flow. The information provided by these studies, however, is generally used for regulatory compliance and not for design. Additionally, local flood studies would be available and may be used for calibration.

Ungaged Sites and Regression Equations

For ungaged sites, the peak flow is most commonly estimated by one of the following methods: USGS regression equations, state regression equations, the rational method, and FEMA Flood Insurance Studies. The rational method is commonly used for areas less than 200 acres to estimate peak flows from urban, rural, or combined areas. It is carried out in accordance with the methods presented in such relevant references as McCuen et al. (2002), HDS-2 and Brown et al. (2009), and *HEC-22*.

Regressions equations give the peak flow Q_T in a river for a given return period T. The equations are based on databases of peak flow obtained from gaged stations. Each peak flow value is associated with several parameters that likely influence the peak flow. Some of these parameters are the drainage area A of the basin contributing to the flow, the mean elevation in the river E, the percent C of bedrock underlying the basin, the percent U of urban area within the basin, and the percent S of storage within the basin. Each influencing factor may be multiplied by a weighing coefficient and a method such as the general least squares method to develop the regression equation. This equation may be of the form

$$Q_T = 10^a A^b E^c (1+0.01C)^d (1+0.01U)^e (1+0.1S)^f \quad (4.10)$$

Where a, b, c, d, e, and f are the regression coefficients that depend on T.

Indeed, Equation 4.10 would be different for the 2-year flow (Q_2), the 5-year flow (Q_5), the 20-year flow (Q_{20}), the 100-year flow (Q_{100}), and the 500-year flow (Q_{500}). Some of the simplest equations are of the form

$$Q_T = 10^a A^b \quad (4.11)$$

These equations are often developed on a regional basis to improve the precision and decrease the scatter. They are extremely useful as they give Q_T, which are fundamental design parameters. However, during the design process it is very important to keep the significant scatter in mind, which is associated with the estimate of Q_T. Also important is to realize that the scatter increases as the return period increases. For example, the precision for Q_{100} is much poorer than the precision for Q_{10}. Figure 53 shows an example of the scatter that can be expected with some of the most effective regression equations. Note that both scales are log scales indicating that the scatter is very large. This is why it is always much more reliable if at all possible to use the flow Q_T given by analyzing a gage station on the river.

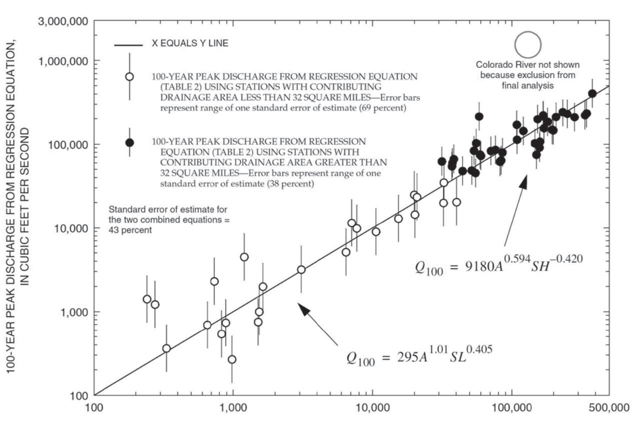

FIGURE 53 Example of comparison between predicted peak flow from regression equation and measured peak flow from a gage station data analysis (Asquith and Slade 1997).

Hydrology Software for Gaged and Ungaged Sites

The software used by the surveyed DOTs for hydrologic purposes include the following: USGS StreamStats, USACE HEC-HMS, USDA NRCS WinTR-55, WMS, and USACE HEC-SSP. A brief description of the capabilities of each software package is included in Table 5.

TABLE 5
HYDROLOGY SOFTWARE PACKAGES ADOPTED IN CURRENT PRACTICE

Software	Description
StreamStats	Web-based Geographic Information System (GIS) application used to create streamflow statistics for gaged and ungaged sites.
HEC-HMS	USACE Hydrologic Modeling System simulates the complete hydrologic processes of dendritic watershed systems. It includes hydrologic analysis procedures such as event infiltration, unit hydrographs, and hydrologic routing in addition to a range of advanced capabilities and tools.
WinTR-55	Tool used to perform accurate hydrological analysis of small watershed systems.
WMS	Watershed Modeling System is a complete program for developing watershed computer simulations. WMS supports lumped parameter, regression, and 2-D hydrologic watershed modeling, and can be used to model both water quantity and water quality.
HydroCAD	Computer-aided design tool used to model storm water runoff. It provides commonly used drainage calculations that include the rational method, SCS, NRCS, SBUH, etc.
HEC-SSP	USACE statistical software package used to perform statistical analyses of hydrologic data. The current version of HEC-SSP performs flood flow frequency analysis is based on Bulletin 17B, *Guidelines for Determining Flood Flow Frequency* (1982).

Another technique to obtain information on the flow is to use the complete system of USGS gages in a state, associate each gage with a longitude and latitude coordinate, and develop an interpolation technique for a location within the area defined by the closest three gages. This is what is done with the freeware TAMU-FLOW for some states (Texas and Massachusetts at present). The location of the 3,116 USGS gages in Texas and neighboring states is shown in Figure 54. Figure 55 shows the map of recurrence interval for any location in Texas developed based on the analysis of 744 gage records and automated in TAMU-FLOOD. The precision of this technique can be assessed by comparing predicted and measured velocities at the location of an existing gage (Figure 56). This figure shows the frequency distribution for the difference between the predicted and observed velocity ratio V_{mo}/V_{100} where V_{mo} is the maximum observed velocity and V_{100} the velocity corresponding to the 100-year flood. As shown in the figure, the mean difference is 3.2% and the standard deviation is 20.8%, which is very satisfactory (Figure 57).

Other Sources of Information

The following are other sources of information useful for hydrologic calculations:

- Soil type information: NRCS Web Soil Survey Web Application (http://websoilsurvey.nrcs.usda.gov/)
- Precipitation information: National Weather Service (http://hdsc.nws.noaa.gov/hdsc/pfds/) and local climate centers.

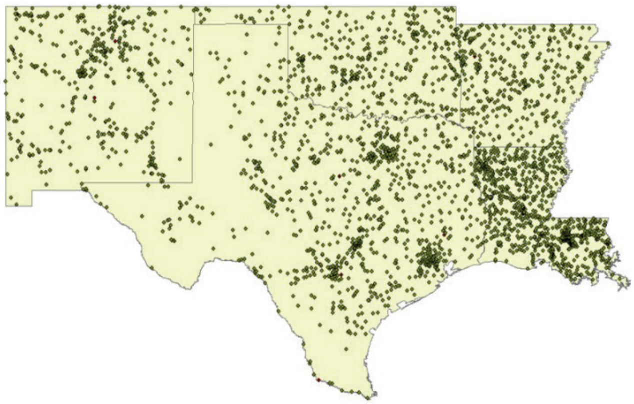

FIGURE 54 Mapping of recurrence interval by using the gages in a state (Briaud et al. 2009), location of the flow gages used for mapping Texas and neighboring states.

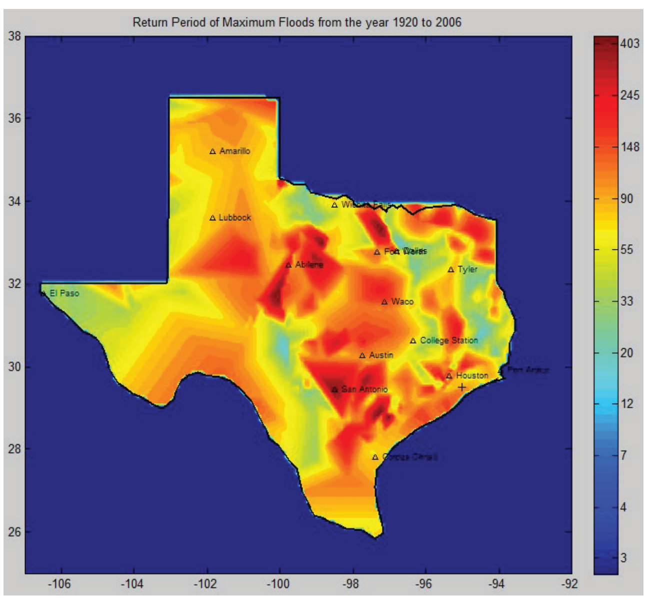

FIGURE 55 Mapping of recurrence interval by using the gages in a state (Briaud et al. 2009), maximum recurrence interval RImo map for Texas for 1920 to 2006.

Additional Considerations

A number of important considerations are highlighted herein when carrying out hydrologic calculations:

- Data considerations: gage data might not be sufficient, or variations might require combined statistical analysis as recommended in Bulletin 17B (USGS 1982).
- Evaluation of flood records: Bulletin 17B explains the importance of evaluating the adequacy and applicability of the flood records that constitute the basis for the flood frequency analysis. In the analysis process, essential considerations are described and include the climatic trends, randomness of events, watershed changes, mixed populations, and reliability of flow estimates.
- Use of storm (flood) hydrograph versus peak flow: the peak flow is the commonly used hydrological design parameter. However, for applications in which the duration of an event or the volume of the discharge are useful (for instance, the duration of an overtopping event), storm hydrographs would be used.

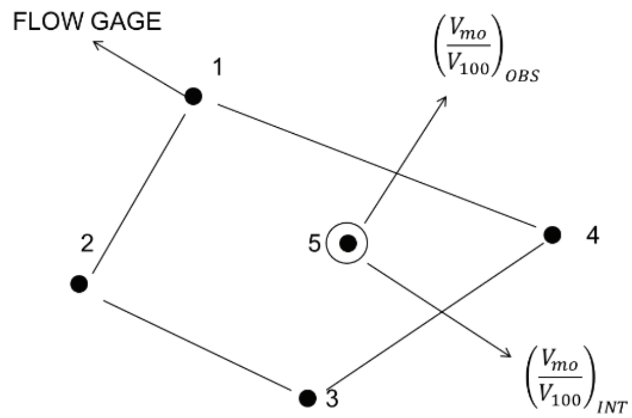

FIGURE 56 Interpolation scheme and precision of the predictions using TAMU flood (Briaud et al. 2009), interpolation scheme among four neighboring flow gages and verification.

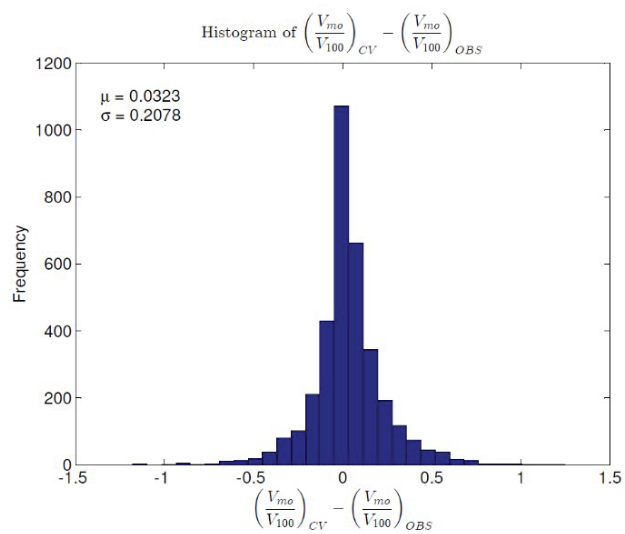

FIGURE 57 Interpolation scheme and precision of the predictions using TAMU flood (Briaud et al. 2009), predicted versus measured V_{mo}/V_{100}.

Coastal Environments

Limited sources for coastal hydrology were provided through the survey responses. The main source of coastal information for tides and currents is the National Oceanic and Atmospheric Administration (http://tidesandcurrents.noaa.gov/).

Additional information sources include the following:

- Jones et al. (2005), *Wave Run-Up and Overtopping*
- USACE (2008), *Coastal Engineering Manual,* EM 1110-2-1100
- USACE (2012), *Hurricane and Storm Damage Risk Reduction System Guidelines*
- Office of Federal Lands Highway (2014), *Project Development and Design Manual* (PDDM; FHWA 2014).

HYDRAULIC METHODS AND CONSIDERATIONS

As explained in chapter two ("Failure Modes" section), overtopping mechanisms and wave action cause surficial erosion. Overtopping leads to the initiation of back erosion mechanisms on the downstream (landward) slope. Wave action leads to damage on the slope that it impacts.

Once the hydrologic data are collected, mainly for the design flood with a chosen return period, the hydraulic work can start. This work consists of, among other things, calculating the water elevation and water velocity that corresponds to the design flood.

Modeling Surface Water Level (For Coastal and Riverine Environments)

Surface water level modeling is an important aspect of design. One-dimensional and two-dimensional modeling packages are available for this purpose. One-dimensional models are used to compute the average depth and velocity for open channel flow. Two-dimensional models are used to compute the water surface profile, the depth, and the velocity across the channel. Further information can be found in many references such as HDS-6 and PDDM.

Based on DOT current practice, the most commonly used hydraulic engineering software packages are HEC-RAS and HY-8 (Table 6).

FLOW DISCHARGE AND VELOCITY EQUATIONS

Design equations are included herein for the purpose of discharge and velocity calculations. Discharge calculations yield the overtopping height (and vice versa) and contribute in damage calculations (chapter six). The design velocities computed here can be compared with the permissible velocity of the embankment material or embankment protection system. This will be discussed further in chapters six, nine, and 10. The design equations included here were obtained from a review of the available literature. They give insights to the importance of the input parameters in embankment design. As mentioned before, only two modes of failure will be discussed here (overtopping and wave erosion). Design equations for the failure modes will be discussed in chapter six.

TABLE 6
HYDRAULIC SOFTWARE PACKAGES FOR COASTAL AND RIVERINE EMBANKMENTS

Software	Description
HEC-RAS	Hydrologic Engineering Centers River Analysis System is used to perform one-dimensional and, recently, two-dimensional steady flow; unsteady flow; sediment transport/mobile bed computations; and water temperature modeling.
HY-8	HY-8 automates culvert hydraulic computations using a number of essential features that make culvert analysis and design easier.

CASE I: FLOOD OR STORM SURGE OVERTOPPING

The flood or surge discharge for an embankment overtopped by water can be calculated using equations for broad crested weirs. Equations are presented herein for riverine and coastal surge.

Riverine Overtopping

Depending on the type of flow (free flow or submerged), the following equations can be used:

– For the free-flow case (low tailwater), the water falls while following the slope contour, creating the following relationship:

$$q = CH^{\frac{3}{2}} \tag{4.12}$$

Where:

$$H = h + \frac{v^2}{2g} \tag{4.13}$$

Where:

– q is the discharge per unit width ft^2/s (m^2/s),

– H is the total head (static + velocity) above the roadway crest in ft (m),

– h is the headwater height above the roadway crest in ft (m),

– v water velocity in ft/s (m/s),

– g is the gravitational acceleration ft/s^2 (m/s^2), and

– C is an experimentally determined discharge coefficient for free flow.

Note that the velocity head can be ignored (applicable for the riverine case) if the approach velocity v is assumed to be very low.

- For the submerged flow case (high tailwater), the difference in elevation between the upstream water level and the tailwater level is not large, the embankment is submerged, and the modified equation is

$$q = CLH_1^{3/2}\frac{C_s}{C} \tag{4.14}$$

Where:

– L is the length of the inundated roadway, and

– C_s is a submergence coefficient.

Values of C *and* C_s

Yarnel and Nagler (1930) were the first to present charts for the determination of discharge coefficients for overtopping flow over railway and roadway embankments. These charts were later modified by a USGS memo ("Computation of Discharge over Highway Embankments," March 16, 1955). A compilation and analysis of vital information on broad-crested weirs was then prepared by Tracy (1957). Kindsvater (1964) presented a detailed study that discusses the theoretical and experimental basis for computing the peak discharge from post-flood field observations. The references for where to find the C and C_s values associated with each study is shown in Table 7.

TABLE 7
DISCHARGE COEFFICIENTS FROM VARIOUS STUDIES

Reference	Computation of Discharge Coefficient C and Submergence Factor C_s
Yarnel and Nagler (1930)	Charts for C and C_s
USGS Memo (1955)	Charts for C and C_s
Kindsvater (1964)	Values for C and C_s
Bradley (1973)	Charts (Figure 58) for C and C_s
Powledge et al. (1989)	C ranges between 1.60 and 2.15 in Metric Units (2.9 and 3.9 in English Units)
	Typical value of 1.9 in Metric Units (3.0 in English Units) that is used for level-crested structures C_s from Bradley (1973)
Richardson (2001)	Charts (Figure 59)

The significant and insignificant factors in the discharge characteristics, based on Kindsvater (1964), are shown in Table 8.

A similar equation for the flow discharge was presented by Petersen (1986) as follows:

$$Q_o = K_u K_t C_r L_s (HW_r)^{1.5} \tag{4.15}$$

Where:

– Q_o is the overtopping discharge in m^3/s (ft^3/s),

– C_r is the overtopping discharge coefficient,

– HW_r is the flow depth above the roadway in m (ft),

– K_t is the submergence factor,

– L_s is the length of the roadway crest along the roadway in m (ft), and

– K_u = 1.0 (English units) and K_u = 0.552 (SI units) (Richardson 2001).

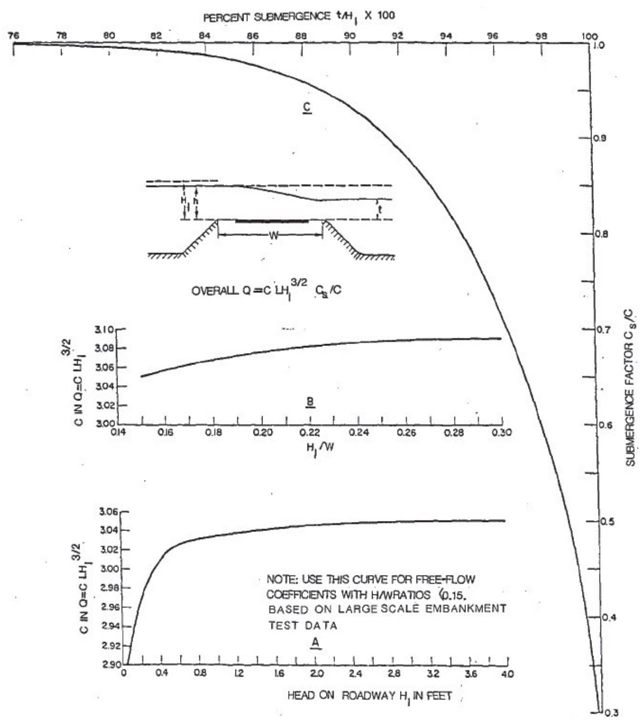

FIGURE 58 Discharge coefficients for overtopping flow over roadway embankment (after Bradley 1973).

TABLE 8
SIGNIFICANT AND INSIGNIFICANT FACTORS INFLUENCING THE OVERTOPPING DISCHARGE

Factors	Significant	Insignificant
Embankment Slope	_	Insignificant except for its effect on the roller on the downstream side
Crest Width	To some extent, as it governs the head loss and the boundary layer thickness at the control section	_
Crest Roughness	To some extent, as it governs the head loss and the boundary layer thickness at the control section	_
Embankment Height	_	Insignificant
Pavement Cross Slope and Shoulder Slope	_	Insignificant

Coastal Surge Overtopping Coefficient

Based on Hughes (2008), it appears appropriate to plug a discharge coefficient value C_f of 0.5443 into the following equation:

$$q = C_f\sqrt{g}\ H^{\frac{3}{2}} \tag{4.16}$$

The reasoning presented is that $C_f \leq 0.5443$ was presented in Chen and Anderson (1987) nomograms, with this coefficient being a function of the upstream head over the crest to the crest width (H/w). The decrease of C_f is insignificant until the head becomes less than 0.5 ft. As a result, "it seems appropriate to be slightly conservative" (Hughes 2008) (Figures 59 and 60; Table 8).

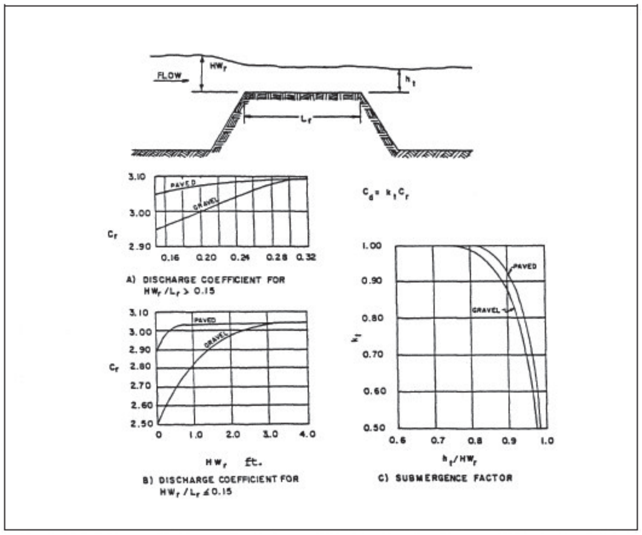

FIGURE 59 Discharge coefficients for overtopping flow over roadway embankments (after Petersen 1986).

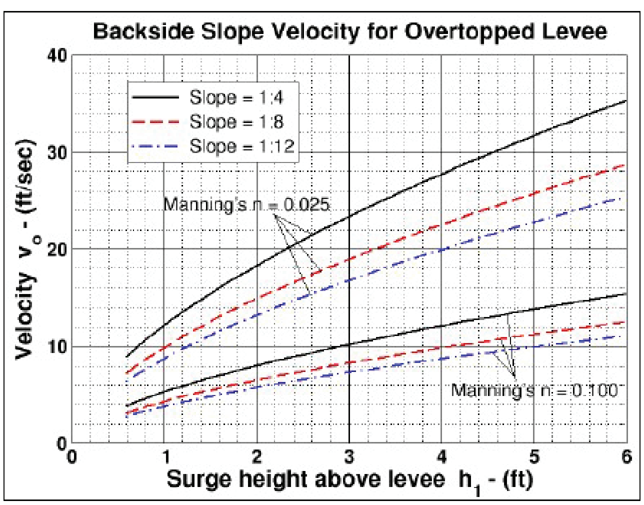

FIGURE 60 Variation of downstream slope velocity as a function of surge height for different Manning coefficients (after Hughes 2008).

Velocity Calculations

Manning's equation for flow resistance is presented as follows:

$$v = \frac{1}{n}R^{\frac{2}{3}}S_f^{\frac{1}{2}} \tag{4.17}$$

Where:

– v is the velocity in m/s,

– R is the hydraulic radius in m,

– S_f = sin θ where θ is the slope angle of the river bottom, and

– n is Manning's coefficient ($s.m^{-0.33}$) ranging from 0.01 for a smooth clay surface to 0.03 for a gravelly surface to 0.05 for a boulder surface.

Because a wide channel with steady, uniform flow is assumed, R becomes the flow depth. By substituting for the depth of flow = q/v and S_f = sinθ (θ being the angle of the river bottom slope), the following equation is obtained:

$$v = (\frac{1}{n}\sqrt{sin\theta})^{\frac{3}{5}}q^{\frac{2}{3}} \tag{4.19}$$

Where:

– q is the flow per unit length of embankment, and

– n is Manning's coefficient.

Figures 60 and 61 show the variation of downstream slope velocity and flow thickness, respectively, as a function of increased surge height over the coastal embankment crest. These figures were prepared based on Hughes's (2008) physical model.

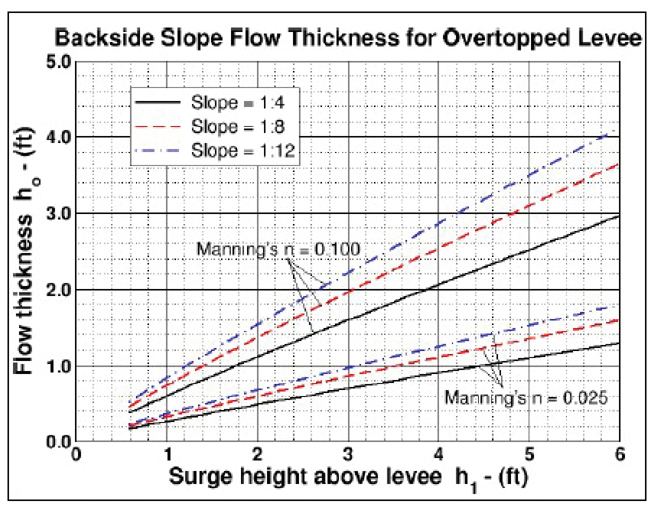

FIGURE 61 Variation of the downstream flow thickness as a function of surge height (after Hughes 2008).

CASE II: WAVE OVERTOPPING

Advances on the determination of overtopping flow parameters have been made based on small-scale and large-scale experiments on levees carried out in Europe (Schüttrumpf et al. 2002; van Gent 2002; Schüttrumpf and Oumeraci 2005). The differences between van Gent's work (the Netherlands) and Schüttrumpf's work (Germany) in 2002 were later reconciled to the possible extent through a joint paper. A summary of the work carried out is presented by Hughes (2008). The main design equations are included herein.

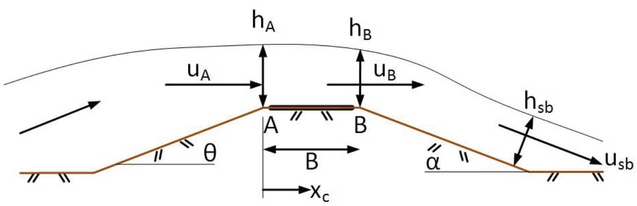

FIGURE 62 Wave overtopping (after Schüttrumpf and Oumeraci 2005).

The velocity and flow depth associated with coastal wave overtopping (Figure 62) can be determined at the following three locations over the embankment (levees):

– Point A: The crest edge on the upstream side (u_A, h_A)

– Point B: The crest edge on the downstream side (u_B, h_B)

– Point SB: The slope bottom of the landward slope (us_b, hs_b)

The three key parameters used as input for the flow velocity and flow depth calculations are shown in Table 9.

TABLE 9
INPUT PARAMETERS FOR VELOCITY AND DEPTH CALCULATIONS FOR WAVE OVERTOPPING

R_c	Levee Freeboard
$R_{u2\%}$	The run-up elevation exceeded by only 2% of the waves, estimated using the run-up formulas of de Waal and van der Meer (1992) or Hughes (2004)
f_F	Friction factor that accounts for frictional energy loss as the overtopping wave travels across the crest and down the protected-side slope

Determination of Input Design Parameter $R_{u2\%}$

Significant Wave Height H_{mo}

This is the primary measure of energy in a sea state and is equal to the average height of the one-third-highest waves.

Wave Run-Up $R_{u,2\%}$

The height of the 2% wave run-up can be estimated based on the following equation (Douglas and Krolak 2008):

$$\frac{R_{u,2\%}}{H_{mo}} = 1.6\, r\vartheta_{op} \qquad (4.20)$$

Where:

– $R_{u,2\%}$ is the run-up level exceeded by 2% of the run-ups in an irregular sea,

– H_s is the significant wave height near the toe of the slope, and

– r is a roughness coefficient (r = 0.55 for stone revetments) and ϑ_{op} is the surf similarity parameter defined as:

$$\vartheta_{op} = \frac{\tan\theta}{\sqrt{\frac{2\pi H_S}{gT_p^2}}} \qquad (4.21)$$

Where:

– θ is the slope angle (Figure 63),

– H_s is the significant wave height,

– T_p is the wave peak period, and

– g is the gravitational acceleration.

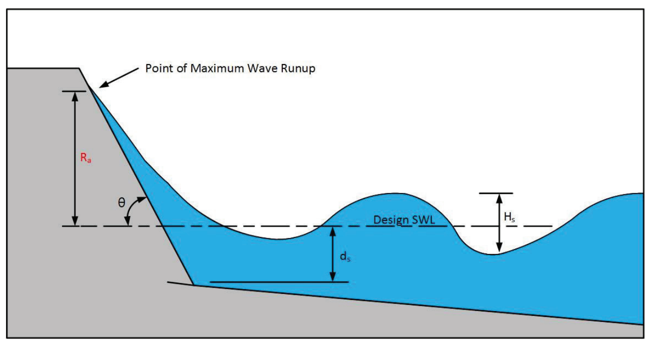

FIGURE 63 Wave run-up sketch (after Douglas and Krolak 2008).

Equations 4.22 through 4.31 can be used to estimate the wave overtopping peak velocity and associated flow depth over an embankment that is exceeded by only 2% of the incoming waves.

Crest Edge on the Upstream Side (Point A in Figure 62)

$$\frac{h_{A2\%}}{H_S} = C_{Ah2\%}\left[\frac{R_{u,2\%} - R_C}{H_S}\right] \qquad (4.22)$$

Where $C_{Ah2\%}$=C_2 tanθ

$$\frac{u_{A2\%}}{\sqrt{gH_S}} = C_{Au2\%}\sqrt{\frac{R_{u,2\%} - R_C}{Hs}} \qquad (4.23)$$

Where:

– $h_{A2\%}$ is the peak flow depth exceeded by 2% of the waves,

– H_s is the significant wave height [$= H_{mo}$],

– $C_{Ah2\%}$ is an empirical depth coefficient (Table 10),

– C_2 is a constant and θ is the flood-side slope angle,

– $u_{A2\%}$ is the depth-averaged peak velocity exceeded by 2% of the waves.

– g is the gravitational acceleration, and

– $C_{Au2\%}$ is an empirical velocity coefficient (Table 10).

TABLE 10
SUMMARY OF EMPIRICAL COEFFICIENTS FOR FLOW PARAMETERS

Empirical Coefficient	Schüttrumpf	van Gent
$CAh2\%$ (depth)	0.33[1,2] and 0.22[4]	0.15[2,3]
$CAu2\%$ (velocity)	1.55[1] and 1.37[2]	1.30[2,3]
$CAh50\%$ (depth)	0.17[1,4]	–
$CAu50\%$ (velocity)	0.94[1,4]	–

Modified after Hughes (2008)
[1] Schüttrumpf et al. (2002)
[2] Schüttrumpf and van Gent (2003)
[3] van Gent (2002)
[4] Schüttrumpf and Oumeraci (2005)

According to Hughes (2008), the recommended coefficient values from Table 10 are $C_{Ah2\%} = 0.22$ [which is the most recent value provided by Schüttrumpf and Oumeraci (2005)] and $C_{Au2\%} = 1.55$ (1.37 was believed to be a typo).

In conclusion, the significant parameters in determination of case a. flow parameters are the flood-side slope and the wave parameters used in the estimation of $R_{u,2\%}$.

Crest Edge on the Upstream Side (Point B in Figure 62)

As the overtopping waves flow across the embankment crest, their height and velocity decrease as a result of the surface friction. The following equations can be used to estimate the height and velocity at any location along the crest width:

$$h_{B2\%} = h_{A2\%} \exp\left(-C_3 \frac{x_c}{L}\right) \tag{4.24}$$

Where:

– L is the crest width,

– x_c is the distance along the crest from the flood-side edge (Figure 61), and

– C_3 is an empirical coefficient (Table 11).

TABLE 11
C_3 VALUES BASED ON LITERATURE

Reference (based on 2% exceedance levels)	C_3 Value
Schüttrumpf et al. (2002)	0.89 for TMA[1] spectra
	1.11 for natural spectra
Schüttrumpf and van Gent (2003)	0.40[2] and 0.89
Schüttrumpf and Oumeraci (2005)	0.75[3] for irregular and regular waves

[1] TMA spectrum: a spectrum created by combining the first three letters of the three wave data sets (Texel, MARSEN, and ARSLOE).
[2] Used if van Gent's (2001) method for estimation of wave run-up was used.
[3] Used if de Waal and van der Meer (1992) or Hughes (2004) was used.

The velocity associated with $h_{B2\%}$ can be calculated as follows:

Where $C_{Ah2\%} = C_2 \tan\theta$

$$u_{B2\%} = u_{A2\%} \exp\left(-\frac{x_c f_F}{2h_{B2\%}}\right) \tag{4.25}$$

$$f_F = \frac{2gn^2}{h^{1/3}} \tag{4.26}$$

Where:

– f_F is Fanning factor for the embankment surface,

– $h_{B2\%}$ is the flow depth at that location of the crest previously obtained through Equation 4.24,

– n is Manning's coefficient,

– h is flow depth in meters, and

– $u_{A2\%}$ is calculated at point A.

Some f_F values based on experimental results from the available literature are presented in Table 12.

TABLE 12
f_F VALUES BASED ON EXPERIMENTAL RESULTS FROM AVAILABLE LITERATURE

Reference	Surface Material	Fanning Friction Factor f_F
Schüttrumpf et al. (2002)	Wood fiberboard	0.0058
	Bare compacted clay surface	0.01
Schüttrumpf and Oumeraci (2005)	Range of values on the protected side slope	0.02 (smooth slopes)
Cornett and Mansard (1994)	Rough revetments and rubble mound slopes	0.1–0.6
Hughes (2008)	Grass-covered slopes	≈ 0.01

No published values of Fanning's coefficient exist for armored alternatives. A fist attempt to approximate the Fanning factor was presented by Henderson (1966). In this approximation, a knowledge of Manning's coefficient is required for a particular armoring product or surface slope. This equation should be used with caution because it has not yet been proven.

Slope Bottom of the Landward Slope (Point sb in Figure 62)

According to Hughes (2008), two theoretical expressions exist to calculate the velocity at the bottom of the downstream slope. Both formulas give relatively close answers. Schüttrumpf and Oumeraci (2005) presented an iterative solution. An explicit one was presented by van Gent (2002) and is shown here:

$$u_{sb2\%} = \frac{K_2}{K_3} + K_4\, EXP\left(-3\, K_2 K_3{}^2 s_b\right) \tag{4.27}$$

With:

$$K_2 = (g \sin \alpha^{\frac{1}{3}}) \tag{4.28}$$

$$K_3 = \left[\frac{f_F}{2}\frac{1}{h_{B2\%} u_{B2\%}}^{1/3}\right] \tag{4.29}$$

$$K_4 = u_{B2\%} - \frac{K_2}{K_3} \tag{4.30}$$

Where:

– α is the angle of the downstream slope,

– s_b is the distance down the slope from the crest edge,

– $h_{B2\%}$ is the flow depth at the crest edge, and

– $u_{B2\%}$ is the flow velocity at the crest edge.

The flow thickness down the slope $h_{sb2\%}$ can be estimated using the following equation:

$$h_{sb2\%} = \frac{h_{B2\%}\, u_{B2\%}}{u_{sb2\%}} \tag{4.31}$$

Case III: Wave and Surge Overtopping

Hughes (2008) presented this equation to calculate the discharge associated with this combined case of wave and surge overtopping:

$$Q_{ws} = \frac{q_{ws}}{\sqrt{gH_{mo}{}^3}} = 0.0336 + 0.53\, \frac{-R_c}{H_{mo}}^{1.58}\ ;\ R_c < 0 \tag{4.32}$$

Where:

– Q_{ws} is the average combined wave and surge discharge,

– H_{mo} is the energy-based significant wave height, and

– R_c is the freeboard, which is negative for this formula.

Figure 64 shows a plot of the data of the combined average discharge versus relative freeboard. This equation was derived based on total of 27 experiments covering a range of three storm surge elevations exceeding the levee crest and nine irregular wave conditions, and it is to be used strictly for waves shoaling on a 1:4.25 levee flood-side slope. If the slope was milder or steeper, different results would be expected because the seaward slope affects the waves that overtop the levee.

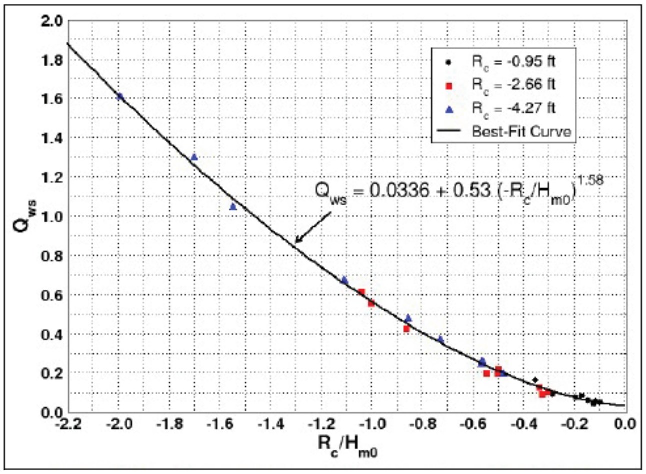

FIGURE 64 Dimensionless combined average discharge versus relative freeboard (after Hughes 2008)

SUMMARY

This chapter presented an overview of the hydrologic and geologic factors that play a role in the design of roadway embankments in riverine and coastal environments during flooding. The impact of a selected recurrence interval on the velocity and water depth was also assessed. The next chapter discusses the geological and geotechnical factors involved in roadway embankment design.

CHAPTER FIVE

GEOTECHNICAL AND GEOLOGICAL FACTORS

INTRODUCTION

The modes of failure identified in chapter two deal with the interaction between the water and the soil. The water represents the "load" and the soil the "resistance." The water effect is characterized by the water velocity v (m/s) and the corresponding hydraulic shear stress τ (N/m^2). The soil resistance to erosion is characterized by the relationship between the erosion rate $\dot{z}$ (mm/h) on one hand and the water velocity v or shear stress τ on the other (Figure 65). This relationship is called the erosion function of the soil; it represents the fundamental behavior of the soil much like the stress strain curve represents the fundamental behavior of the soil when subjected to mechanical loading. The erosion function is nonlinear and can be measured by erosion testing in the laboratory or in the field. Some of the erosion tests available are described later in this chapter.

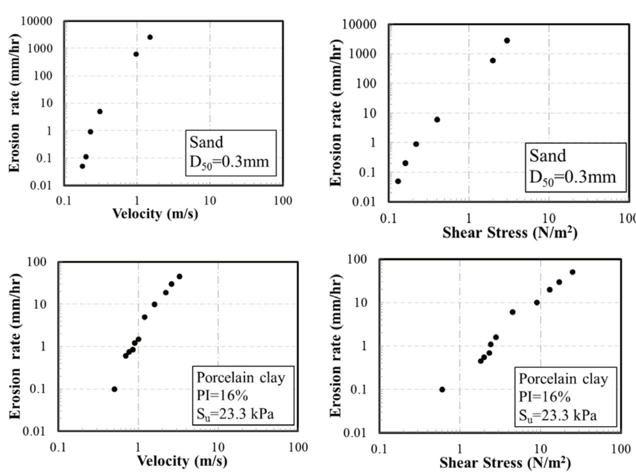

FIGURE 65 Erosion functions for a sand and for a clay.

GEOTECHNICAL CONSIDERATIONS

Critical Velocity

As discussed previously, the erodibility of a soil is described by the erosion function. The erosion process does not start until the velocity is large enough to initiate erosion of the soil particles. This threshold of erodibility is called the critical velocity v_c and plays a very important role in erosion engineering. Indeed, if this velocity can be precisely determined, any lower velocity will not create erosion. Figure 66 shows a relationship between the critical velocity and the mean grain size D_{50}, which is the soil grain size for which 50% by weight of the soil grains is larger than D_{50}. As Figure 66 illustrates, for sands and gravels there a good relationship between vc and D_{50} (Briaud 2013). The following relations are derived in SI units. If needed, the values in metric units can be calculated using the conversions table.

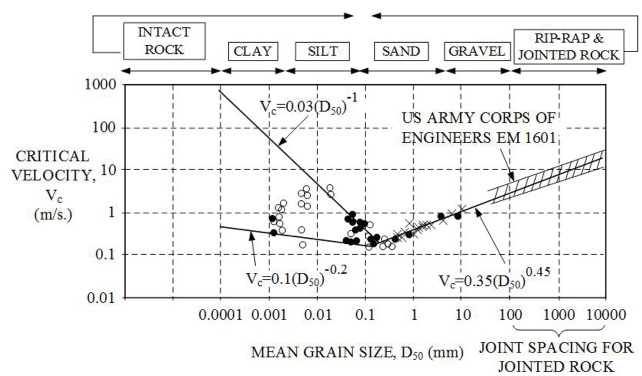

FIGURE 66 Critical velocity and mean grain size.

For sand and gravel
$$v_c(m/s) = 0.35\left(D_{50}(mm)\right)^{0.45} \quad (5.1)$$

However, this relationship breaks down for silts and clays because the behavior of the particles of sand and gravel is controlled by their weight, which is not the case for silts and clays. For silts and clays, the electrostatic and electromagnetic forces that exist between such fine particles become more important than the gravity force or weight of the particle. As a result, the relationship with grain size is scattered and the critical velocity can only be measured on a site-specific basis at this time. Upper and lower bounds can be used to bracket the critical velocity, as shown in Figure 66.

Upper bound for silt and clay

$$v_c(m/s) = 0.03\left(D_{50}(mm)\right)^{-1} \quad (5.2)$$

Lower bound for silt an clay

$$v_c(m/s) = 0.1\left(D_{50}(mm)\right)^{-0.2} \quad (5.3)$$

Erosion Function and Soil Classification

The most effective way to obtain the erosion function is to measure it on a site-specific basis by testing samples or by in situ testing. If this preferred approach is not possible, the erosion function can be estimated on the basis of the Unified Soil Classification System by using the erosion category chart shown in Figure 67 (Briaud 2013). To use this chart, the soil that will be eroded is classified according to the Unified Soil Classification System to obtain the dual symbol classification indicated in the chart, and the straight line that splits the erosion category zone is selected as an average for that soil. The lower bound or upper bound of that category can be used to be conservative, depending on the erosion problem.

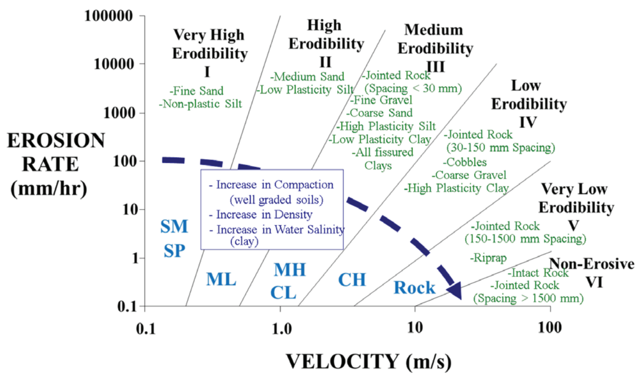

FIGURE 67 Erosion function and soil classification (Briaud 2013).

Example Process for Calculating Erosion Depth

The following process shows how the erosion function can be used for simple calculations by hand to estimate the erosion depth. The steps are listed as follows:

1. Obtain the erosion function for the soil that may be eroded by the flowing water. This can be done by erosion testing (Figure 65) or by using the erosion category chart based on soil classification.

2. Obtain the velocity hydrograph for the flood to be considered. This is the relationship between the velocity of the water and time. Riverine floods usually last a few days. However, a hurricane that makes landfall may create a surge and associated overtopping that lasts only a couple of hours.

3. Decompose the velocity hydrograph into a series of constant velocity steps.

4. For each constant velocity step, find the erosion rate $\dot{z}_i$ (mm/h) that corresponds to the velocity v_i on the erosion function.

5. Using that erosion rate, find out how much erosion will take place during the time increment corresponding to the velocity step.

$$\Delta z_i = \dot{z}_i \times \Delta t_i \tag{5.4}$$

6. Repeat steps 4 and 5 for all subsequent velocity time steps and calculate all the erosion depth increments corresponding to all velocity time steps.

7. The final erosion depth Z is:

$$Z = \sum \Delta z_i = \sum \dot{z}_i \times \Delta t_i \tag{5.5}$$

The following is an example of this simple step-by-step procedure applied to embankment overtopping. Figure 68*a* shows an embankment being overtopped during a major flood. The erosion function measured in an erosion test performed on a sample from the embankment is given in Figure 68*b*; it shows that the soil is very erosion resistant as the critical velocity is 4 m/s (13.12 ft/s). The velocity hydrograph at the bottom of the downstream side of the embankment where the water goes the fastest is given in Figure 68*c*. It shows that the flood is lasting about 2.5 days, with velocities reaching 12 m/s (39.37 ft/s) at the flood's peak. The calculations proceed according to the steps described previously.

The smooth velocity hydrograph is discretized into time increments of 3 hours each. For each increment, the velocity is constant and as long as the velocity is lower than the critical velocity of 4 m/s (13.12 ft/s), no erosion occurs. For each velocity step, the calculations are simple. For example, during the fifth increment beyond the start of the erosion process, the velocity is 10 m/s (32.81 ft/s) for 3 hours. The erosion function gives an erosion rate of 55 mm/h (2.57 in./h) for a velocity of 10 m/s (32.81 ft/s). Therefore, the erosion depth during these 3 hours becomes larger by 55 mm/h x 3 h = 165 mm (6.5 in.). By following this reasoning from the beginning of the hydrograph all the way to the end, the erosion depth can be calculated and is shown in Figure 68*d* as 1.8 m (5.91 ft).

Erosion Tests

A soil's erodibility can be tested using many different laboratory and in situ tests, which have been developed at an increasing rate mostly over the past 25 years. The laboratory erosion tests include the following:

- Jet Erosion Test (JET)
- Hole Erosion Test (HET) and the previous version of it, the Pin Hole test

- Rotating Cylinder Test (RCT)
- Erosion Function Apparatus (EFA) tests and similar devices (e.g., Sedflume).

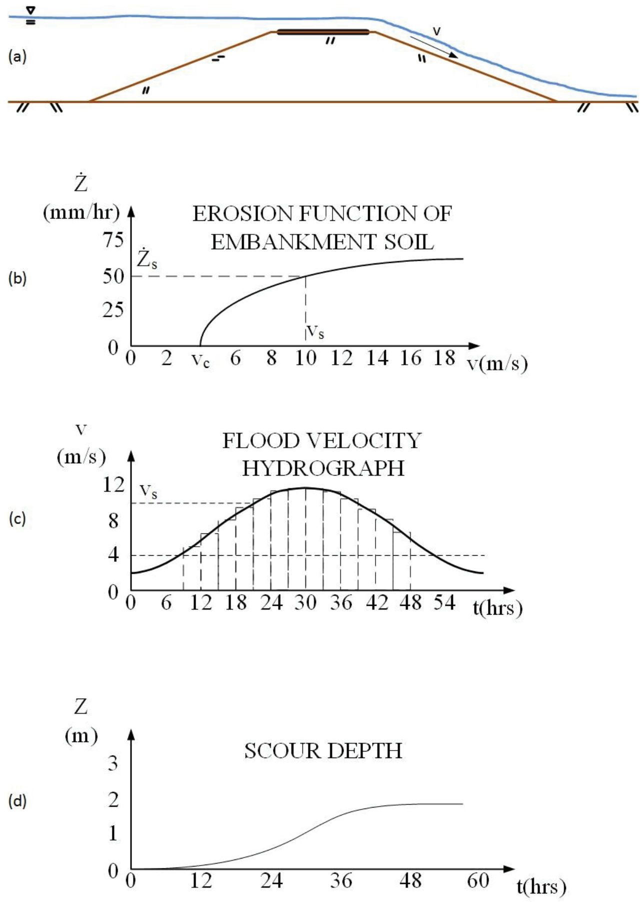

FIGURE 68 Example of the erosion depth calculation.

Jet Erosion Test (JET)

The JET is a laboratory test that can be credited to Greg Hanson of the U.S. Department of Agriculture (Hanson and Cook 2004; USSD 2011). The JET (Figure 69) can be performed in the laboratory on a soil sample or in the field on the ground surface. It consists of directing a stationary water jet at a given velocity perpendicular to the soil surface and recording the depth of the hole made by the jet as a function of time to obtain an erosion rate. The result of the test consists of a curve linking the depth of the hole to the time of jetting. Hanson used a linear relationship to describe the data linking the erosion rate to the net shear stress above critical and called the slope of that line K_D, which he named the erosion coefficient:

$$\dot{z} = K_D\left(\tau - \tau_c\right) \tag{5.6}$$

He went on to classify soils according to their K_D value, as shown in Figure 70.

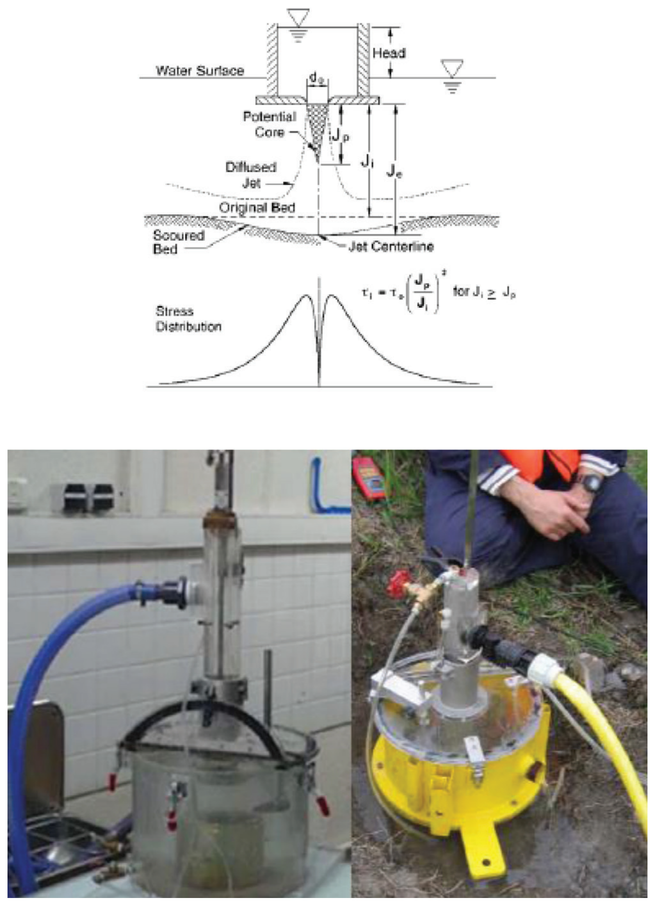

FIGURE 69 Jet erosion test (Hanson and Cook 2004).

Hole Erosion Test (HET)

The HET is a laboratory erosion test that evolved from the older Pin Hole test and can be credited to Robin Fell in Australia (Lefebvre et al. 1984; Wan and Fell 2004; Wahl 2009; Benahmed and Bonelli 2012). The test (Figure 71) consists of drilling a 6-mm (0.24-in.) diameter hole through a soil sample and forcing water to flow through the hole at a chosen velocity while recording the increase in diameter of the hole as a function of time to obtain an erosion rate. The test results link the mass erosion rate to the net shear stress above critical. The equation used is linear:

$$\dot{m} = C_e\left(\tau - \tau_c\right) \tag{5.7}$$

The parameter C_e is called the erosion coefficient. The erosion rate index is then defined as

$$I_{HET} = -\log_{10}\left(C_e(s/m)\right) \tag{5.8}$$

Wan and Fell went on to propose some erosion categories based on I_{HET} (Figure 72).

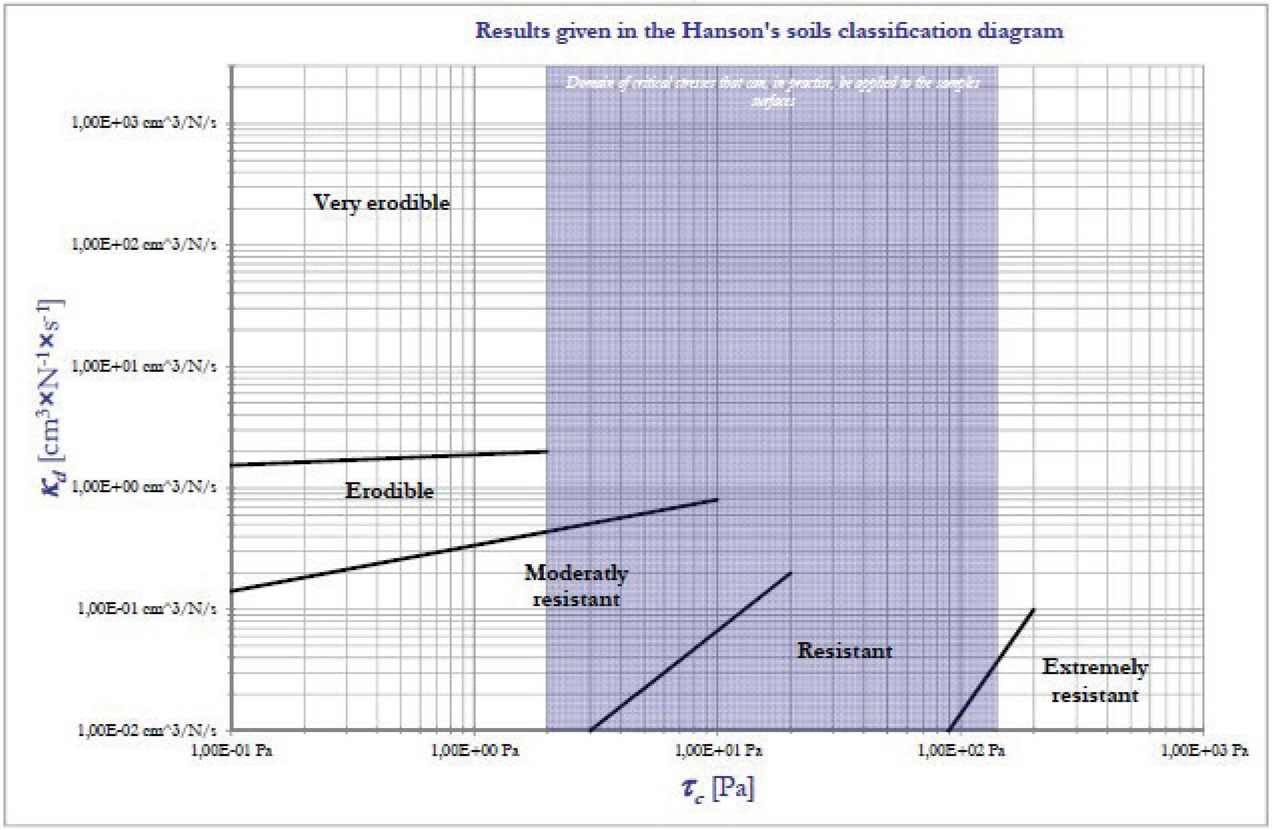

FIGURE 70 Jet erosion test: Hanson's classification according to the erosion coefficient.

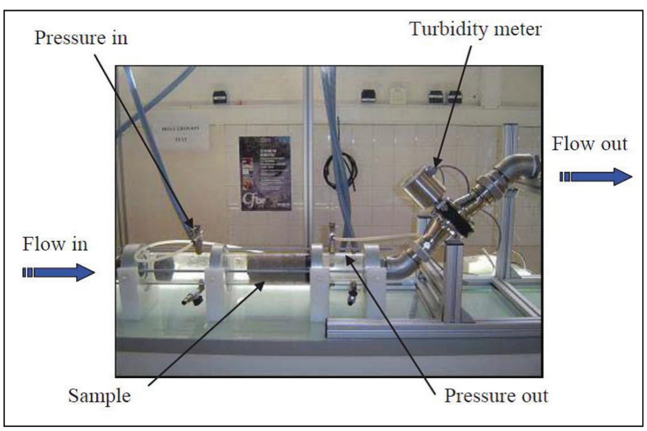

FIGURE 71 Hole erosion test.

Group Number	Erosion Rate Index, I_{HET}	Description
1	< 2	Extremely rapid
2	2 – 3	Very rapid
3	3 – 4	Moderately rapid
4	4 – 5	Moderately slow
5	5 – 6	Very slow
6	> 6	Extremely slow

FIGURE 72 Hole erosion test (continued).

Rotating Cylinder Test

The Rotating Cylinder Test is a laboratory erosion test that has been in existence for a longer time than other tests (Moore and Masch 1962; Chapuis and Gatien 1986; Henderson 1999; Kerr 2001; Sheppard et al. 2005; Bloomquist et al. 2012). It consists of placing a soil sample in a chamber filled with water and rotating the chamber at a speed sufficient to entrain the water up to a chosen velocity. The water erodes the sample, and the decrease in the weight of the sample versus time gives the average erosion rate.

EFA Test and Similar Devices

The EFA test developed in 1991 (Figure 73) and the development of associated bridge scour design guidelines can be credited to Briaud (Briaud et al. 2001; Briaud 2013). Others have also worked on this type of device (McNeil, Taylor, and Lick 1996; Roberts et al. 2003; Crowley et al. 2012). The EFA test is a laboratory erosion test that consists of pushing a soil sample through the bottom of a conduit only as fast as the water flowing over it is eroding it. The erosion rate corresponds to the rate at which the piston is pushing the sample upward. It is recorded for each velocity and gives the erosion function point by point (erosion rate versus velocity or shear stress curve). The equations used for the erosion function in this case are

$$\dot{z} = a\left(v - v_c\right)^b \tag{5.9}$$

$$\dot{z} = \alpha\left(\tau - \tau_c\right)^\beta \tag{5.10}$$

Based on many tests performed over the past 25 years, Briaud proposed an erosion classification and related the soil classification to the erosion classification (Figure 67).

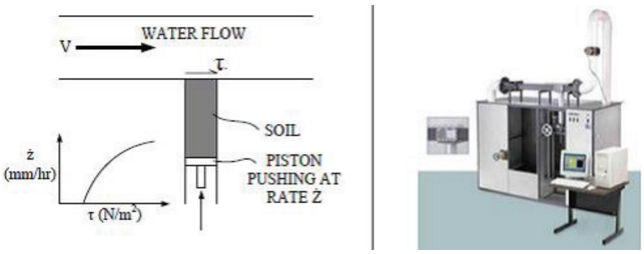

FIGURE 73 EFA test (Briaud 2001).

The field erosion tests are

- In Situ Scour Testing Device (ISTD) from FHWA
- In Situ Scour Evaluation Probe (ISEP) from North Carolina State University
- Borehole Erosion Test (BET) from Texas A&M University
- Pocket Erodometer Test (PET) from Texas A&M University.

In Situ Scour Testing Device

The ISTD is an in situ erosion device being developed by FHWA at the Turner–Fairbanks Highway Research Center. It is a reverse jet test where the water is sucked upward from the outside to the inside of a vertical pipe placed in a borehole. In this process, the tool erodes the bottom of the borehole.

In Situ Scour Evaluation Probe

The ISEP test uses a jetting probe that penetrates into the soil under its own weight at a recorded erosion rate for a given jet velocity. It is credited to Gabr (Gabr et al. 2013; Caruso and Gabr 2010). Gabr follows the work of Hanson and uses the equation

$$\dot{z} = K_D\left(\tau - \tau_c\right) \tag{5.11}$$

Pocket Erodometer Test

The PET (Figure 74) was proposed by Briaud, Bernhardt, and Leclair (2012) as a very simple portable device to test the surface of a sample in the field as the sample is extruded or in the lab before more advanced erosion testing is performed. It consists of using a repeated water jet impulse at 8 m/s (26.25 ft/s) aimed perpendicular to the sample surface, placed 50 mm from that surface and applying 20 repetitions of that jet. Once the test is completed, the depth of the hole generated in the sample surface is measured. Briaud et al. (2013) merged the results of the PET to their erosion classification by adding the depth of the PET hole on the erosion classification chart (Figure 75).

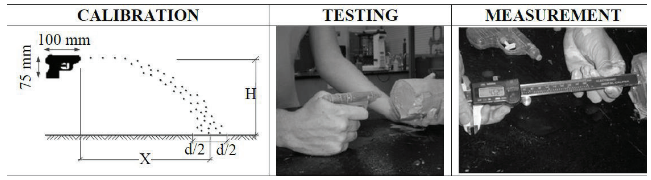

FIGURE 74 Pocket erodometer test (Briaud et al. 2012).

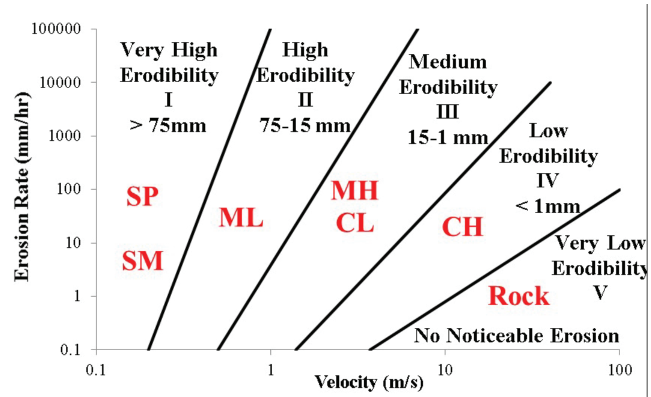

FIGURE 75 Pocket erodometer test chart (Briaud 2013).

Borehole Erosion Test

Briaud (Briaud et al. 2014) developed the BET. This in situ erosion test (Figure 76) consists of recording the increase in a borehole's diameter as the water circulates in it. The test is performed by drilling the borehole to a chosen depth, removing the drilling tool and rods, lowering the diameter measuring tool (borehole caliper) to get a zero diameter reading profile, lowering the rods and bit into the open hole, circulating the water at a chosen velocity for 10 minutes, removing the rods, and lowering the borehole caliper again to take a second diameter measurement. The difference in radius divided by the time of flow gives the erosion rate of all the layers within the borehole depth for the chosen velocity (and, therefore, shear stress). The test is repeated at different velocities and point by point the erosion function of each layer is obtained in one test.

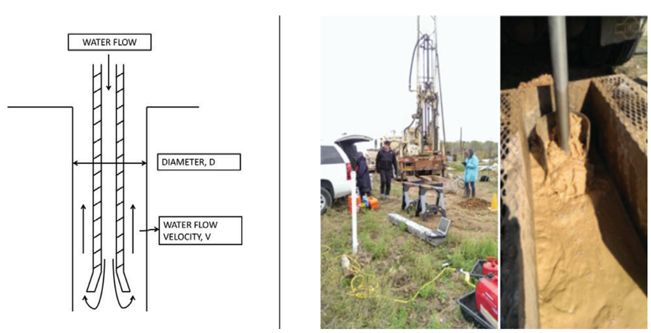

FIGURE 76 Borehole erosion test (Briaud et al. 2014).

GEOLOGICAL CONSIDERATIONS

Geological considerations are an important part of a roadway embankment design to minimize flooding damage. Engineer-

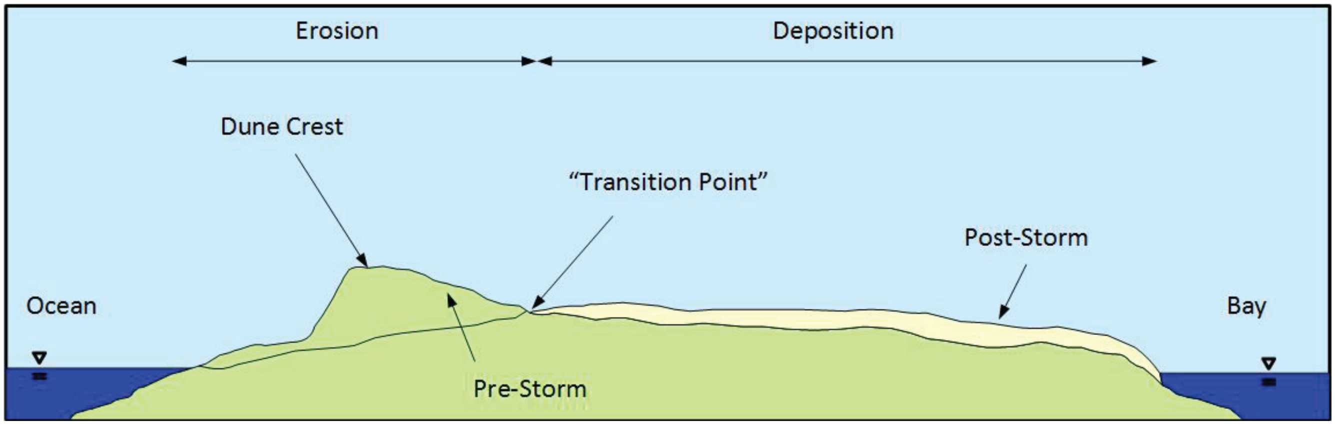

FIGURE 78 Sketch for sand erosion and deposition processes on a barrier island resulting from overwash (Douglas and Krolak 2008).

ing geology brings a big-picture understanding of the surroundings of an embankment and river setting. Within the engineering geology framework, it is necessary to understand the geomorphology of rivers and the impact it has on nearby embankments. Throughout the estimation of the hydrologic input parameters, factors related to drainage basin, stream channel, and flood plain characteristics become essential. The discharge that determines the overtopping height and duration is controlled by the river cross section in addition to meteorological characteristics. The flow velocity is an important design parameter that is controlled by the slope and basin roughness. Other processes affect the damage during flooding. Some of these factors include meandering potential in riverine environments, and erosion and deposition processes in the coastal environments. Those two processes are important considerations in deciding on the location of a roadway embankment, or the recommended measures to mitigate damage.

Meandering Potential

Rivers are active systems; meanders can move laterally several meters per year. This lateral migration of the main channel affects bridges, embankments, and other structures that straddle the river. It is important to predict future meander movements and to design remedial measures or move the structure. An example of meandering is shown in Figure 77 where the average meander rate is 4m/year. Many have contributed to the advancement of knowledge in this field, including Brice (1974), Hickin and Nanson (1984), Hooke (1984), Lagasse et al. (2001), and de Moor et al. (2007).

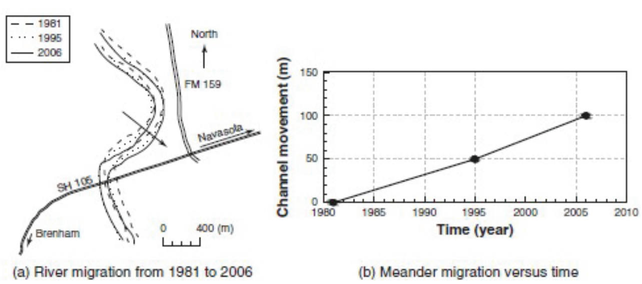

FIGURE 77 Measured migration of the meander over a 25-year period, the Brazos Meander Case History (after Briaud 2013).

Briaud et al. (2007) developed the MEANDER method to predict the movement of a meander over time. First, the initial geometry of the river is described by fitting circles to the meander bends and placing straight-line tangents to the circles between circles. Second, the erosion function of the river banks is input. This can be done by using the results of EFA tests or by using the erosion classification charts of Figure 67 adjusted for the presence of vegetation, trees, or other erosion-retarding layers. Third, the velocity hydrograph is input from measurements at a nearby gage station. Fourth, the circles describing the meanders are moved according to erosion rules developed through a series of very large-scale laboratory meander experiments (in sand and then in clay) as well as numerical simulations (Briaud 2013). This leads to a prediction of the location of the river after the period of time corresponding to the hydrograph.

Erosion and Deposition

As a result of over-wash, erosion and deposition environments can form on barrier islands. As shown in Figure 78, erosion naturally occurs from the ocean side of an island and deposition occurs on the land side. The frontal dunes generally have the highest elevations on the island. In a storm event, these dunes are eroded or possibly overtopped, and the eroded sand can be pushed back through the island into the bay area.

The coastal embankment could lie within the erosion zone or the deposition zone. If it is located in the erosion zone, it would be subjected to extreme wave attack; consequently, it could suffer more damage, depending on the available protection means. On the other hand, if a coastal embankment is located within the deposition environments, it would be found undamaged under a layer of sand that could be scraped off.

DESIGN CONSIDERATIONS FOR FAILURE MODES

The design of flood-prone roadway embankments includes the consideration of the different failure modes explained in chapter two. Design considerations for failure modes resulting from overtopping and wave action were considered in previous chapters—within the limits of existing knowledge and practice. This section covers geotechnical considerations to minimize damage from through-seepage, underseepage, softening by saturation (rapid drawdown condition), overtopping, and lateral sliding. Through-seepage and underseepage may cause internal erosion and piping, which would lead to failure. Internal erosion and piping depend on the material characteristics. Softening by saturation results from seepage through the embankment. Lateral sliding could be caused by softening of the embankment-soil interface and consequent loss of friction. This could also result from damage caused by other processes on the downslope and toe of the embankment, which would decrease the embankment stability. The discussion of the geotechnical aspect of these failure modes is carried out in light of knowledge pertinent to dams and levees.

Internal Erosion Phenomenon

Internal erosion is the reason behind an estimated 46% of earth dam failures, half of which occur during the first filling of the reservoir (Fell and Fry 2005). In general, through-seepage becomes a concern when the embankment is made up of soils subject to internal erosion. The handling of the internal erosion phenomenon is still based primarily on engineering judgment and experience. Although guidelines and publications exist, much remains to be studied in this field.

For internal erosion to occur, the following conditions are required:

1. A seepage path and a source of water.
2. Erodible material that can be carried by the seepage flow within the flow path.
3. An unprotected exit from which the eroded material may escape.
4. For a pipe to form, the material must be able to form and support the roof of the pipe.

Four different phenomena can lead to internal erosion of an embankment, as shown in Figure 79:

1. Backward erosion: initiated at the exit point of the seepage path when the hydraulic gradient is too high and the erosion gradually progresses backward, forming a pipe.
2. Concentrated leak: internal to the soil mass; it initiates a crack or a soft zone emanating from the source of water and may or may not progress to an exit point. Erosion gradually continues and can create a pipe or a sinkhole.
3. Suffusion: develops when the fine particles of the soil wash out or erode through the voids formed by the coarser particles. This occurs when the amount of fine particles is smaller than the void space between the coarse particles. If, in contrast, the soil has a well-graded particle size distribution with sufficiently small voids, suffusion is unlikely. Soils are called internally unstable if suffusion takes place and internally stable if particles are not eroding under seepage flow.
4. Soil contact erosion: sheet flow at interfaces between soil types. This may occur, for example, when water seeps down the back face of the core at the interface with the filter and then the stabilizing mass.

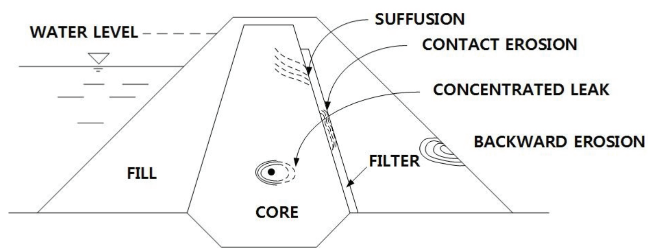

FIGURE 79 Mechanisms of internal erosion failures (after Perzlmaier 2005).

Soils Mostly Susceptible to Internal Erosion

Coarse silt and fine sand are among the most erodible soils. Therefore, embankments that contain significant amounts of such materials will be more prone to internal erosion. Clays in general, and high-plasticity clays in particular, are more resistant to erosion as long as the electrical bonds between particles are not destroyed by chemicals. It appears that some core materials of glacial origin, such as glacial tills, can be particularly susceptible to internal erosion. Sherard (1979) gives a range of gradation of soils that can lead to internal erosion problems (Figure 80).

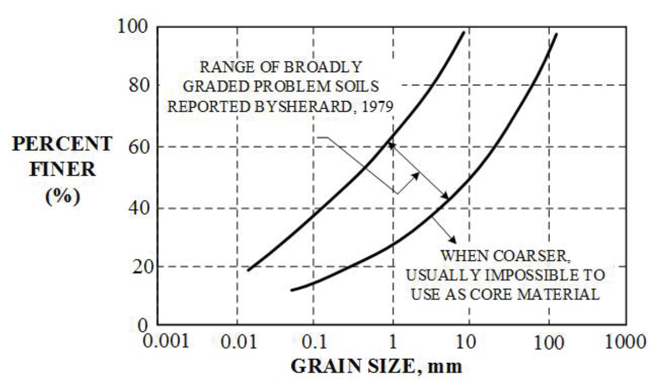

FIGURE 80 Range of problem soils for internal erosion (after Sherard 1979).

The soils most susceptible to suffusion are those where the volume of fines is less than the volume of the voids between coarse particles. In this case, the fines can move easily between the coarse particles and erode away to an exit face. After suffusion, such soils are devoid of fines and become very pervious clean gravel, for example. Fell and Fry (2005) indicate that gap-graded soils and coarsely graded soils with a flat tail of fines (Figure 81) are most susceptible to suffusion.

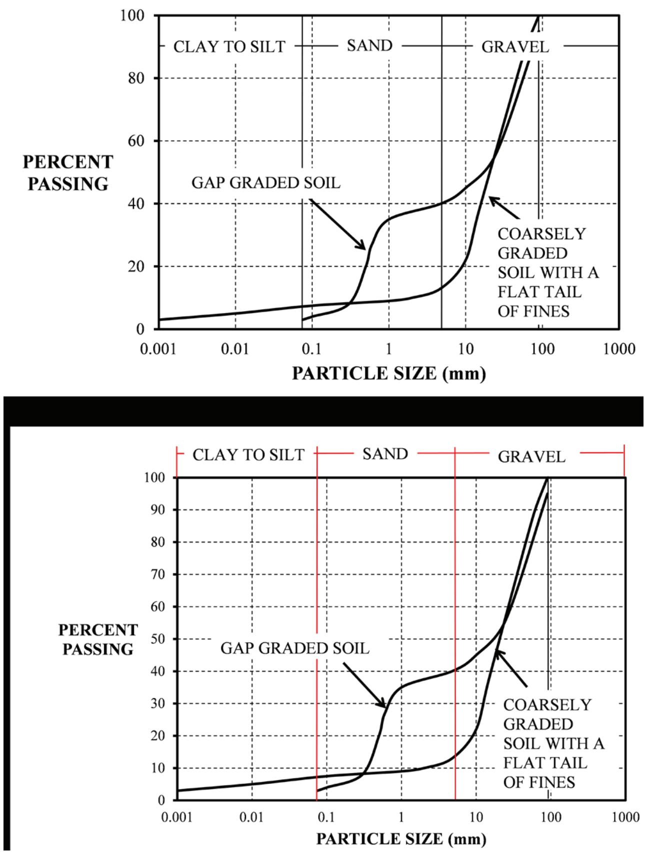

FIGURE 81 Range of problem soils for suffusion (after Fell and Fry 2005).

Criterion to Evaluate Internal Erosion Potential

One of the important criteria for evaluating erosion is to calculate the hydraulic gradient and compare it to the critical gradient. The critical gradient is given by

$$i_{cr} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} \tag{5.12}$$

Values of i_{cr} typically vary in the range of 0.85 to 1.2. In dam applications, the hydraulic gradient depends on many factors including the difference in water level between the upstream and the downstream, the length of the drainage path, and the relative hydraulic conductivity of the various zones. To avoid internal erosion, the target maximum gradient in the flow is kept much lower than the critical value, especially in areas where internal erosion is possible.

For unfiltered exit faces, figure 82 shows ranges of hydraulic gradient values that are associated with the initiation of internal erosion on one hand, and the full development of piping on the other. Generally speaking, there is a trend toward higher-porosity soils beginning to erode at lower hydraulic gradients, even lower than 0.3. Yet soils with plastic fines erode at higher gradients, and gap-graded soils begin to erode at lower gradients than nongap-graded soils with the same fine content.

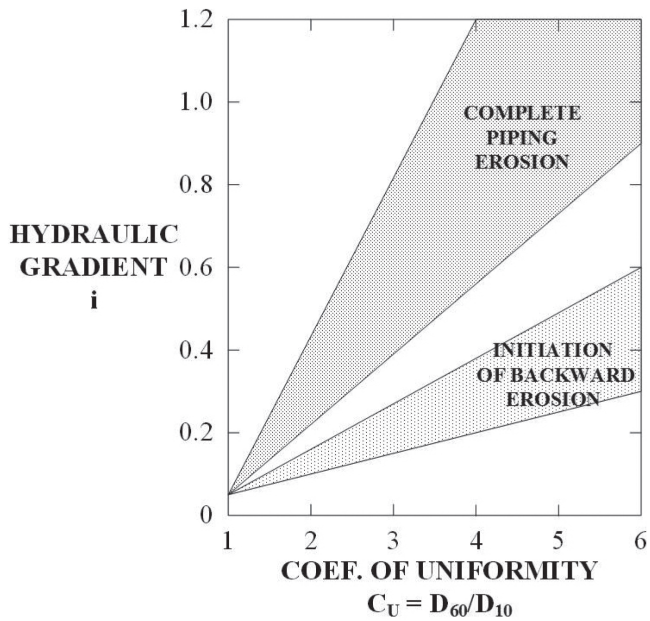

FIGURE 82 Range of hydraulic gradient values associated with internal erosion (after Perzlmaier 2005).

For levee applications, USACE uses a lower-bound value of the critical hydraulic gradient equal to 0.8 and allows a hydraulic gradient of up to 0.5 at the toe of levees, provided a number of conditions are met (USACE 2003a). Another way to address the incipient motion of soil particles in internal erosion problems is to use the concept of critical velocity and charts such as Figure 67. However, these critical velocities were developed from sheet flow tests, and the critical velocity may differ from those initiating internal erosion.

Several methods, based in part on the analysis of the grain size curve, have been developed to evaluate the instability of soils in dams and their sensitivity to the suffusion phenomenon. They include Sherard (1979), Kenney and Lau (1986), Burenkova (1993), and Fell and Fry (2005).

Through-Seepage

Seepage through the embankment, termed through-seepage, leads to a wet downstream slope, as shown in Figure 83. This would weaken the embankment and compromise its stability. Other schemes include the development of internal erosion mechanisms as described in the previous section. To alleviate the problems imposed by the hydrostatic forces, drainage elements can be introduced into the embankment (USACE 2000).

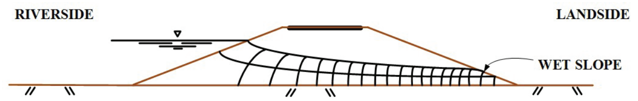

FIGURE 83 Homogeneous embankment section on impervious foundations with emerging seepage on the landside slope (after USACE 2000).

For example, a pervious toe can be integrated into the system, as shown in Figure 84. If both through-seepage and underseepage are anticipated, a combined solution of a pervious toe and a partially penetrating toe trench can be adopted (Figure 85). Other horizontal and inclined drainage options are presented in Figure 86.

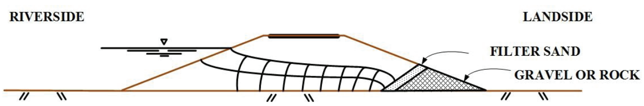

FIGURE 84 Homogeneous embankment section with pervious toe (after USACE 2000).

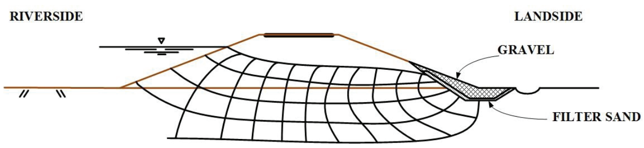

FIGURE 85 Homogeneous embankment section with pervious toe combined with partially penetrating toe trench (after USACE 2000).

Underseepage

Underseepage is a problem generally associated with pervious foundation material. It would impose a severe problem in the following two cases (USACE 2000):

1. A connected pervious substratum underlies an embankment and stretches both upstream and downstream. Erosion could develop within the foundation materials until a void forms under the embankment.
2. A relatively impervious thin-top stratum is present on the downstream side. This could lead to excessive hydrostatic pressures on the downstream side in the underlying, more pervious stratum.

Seepage control solutions in earth foundations are provided in USACE EM 1110-2-1913, EM 1110-2-1901, and EM 1110-2-1914. The methods are namely cutoff walls placed beneath the embankment, riverside blankets, downstream seepage berms (Figure 87), pervious toe trenches (Figure 88), and pressure relief wells. Such measures would alleviate the severity of the seepage conditions.

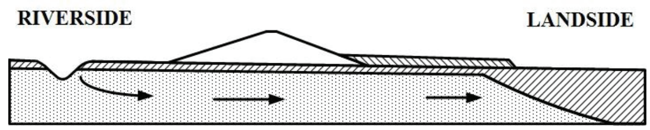

FIGURE 87 Installation of a berm while considering foundation material conditions (USACE 2000).

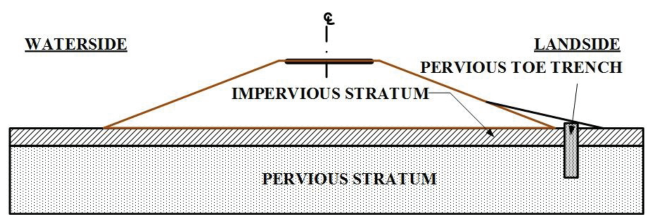

FIGURE 88 Typical partially penetrating pervious toe trench (USACE 2000).

Softening by Saturation (Rapid Drawdown Condition)

Softening by saturation generally occurs after the embankment becomes saturated owing to prolonged exposure to water. This water could come from precipitation, the rising headwater or riverside water level, or inundation during overtopping. The extent of saturation depends on the embankment's permeability and the duration of precipitation, rise in flood level, or inundation. This is accompanied by the embankment's loss of strength and is generally characterized by slope failures and pavement deterioration resulting from the saturation of the subgrade.

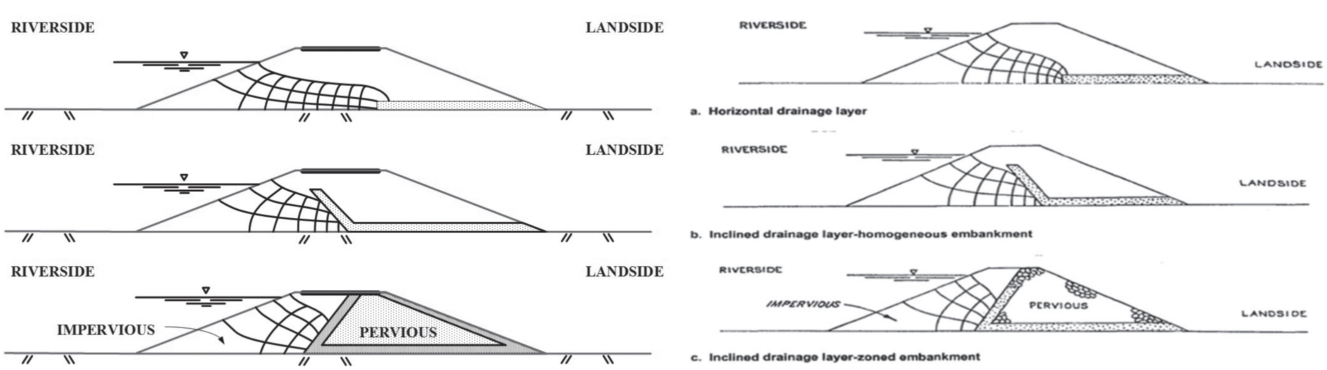

FIGURE 86 Horizontal and inclined drainage layer options (after USACE 2000).

Rapid drawdown is associated with a prolonged flood stage that saturates—by seepage—a major part of the upstream portion of the embankment. The floodwaters then recede faster than the embankment soil can drain, thus resulting in potential instability. For analysis of rapid drawdown, the effective shear parameters are used as recommended by USACE (2000). Two preferred procedures for rapid drawdown analysis are presented in USACE's *Slope Stability Manual* (2003b). To increase stability and prevent slope failure, flatter slopes could be adopted, as well as stability berms. Stability berms also work as a good option in case of emergency to stop any further movement.

Lateral Sliding on Foundations

Figure 89 shows a simplified model for lateral sliding calculations. For sliding stability, the applied force from the upstream water is kept less than the maximum resisting force developed at the embankment-foundation interface by a certain safety factor. The driving force or push P in kilonewtons/meter (kN/m) of embankment length is

$$P = \frac{1}{2}\gamma H^2 \tag{5.13}$$

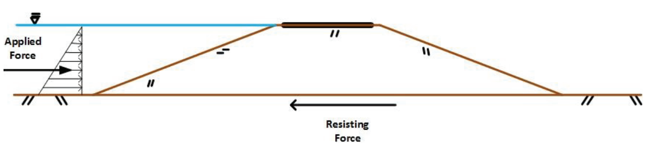

FIGURE 89 Sketch for lateral sliding calculations.

Where γ is the soil unit weight and H is the water depth or, more precisely, the difference in water depth between the two sides of the embankment. The maximum resistance R per unit length of embankment is

$$R = W'\tan\varphi \tag{5.14}$$

Where W is the embankment effective weight per unit length of embankment and φ is the effective angle of friction of the interface at the bottom of the embankment. The factor of safety FS against sliding is

$$FS = \frac{R}{P} \tag{5.15}$$

Overtopping of Embankments

This section presents erosion concepts that are applicable to levees or dikes, and could be applied to roadway embankments. Levees are small dams built along a river or an ocean to prevent the water from inundating the land during floods. The top of the levee is set at a predetermined height corresponding to the water level for a chosen design flood. This flood corresponds to a certain return period, such as a 100-year flood. Similar to the case of the roadway embankments, if the flood exceeds the design return period, water is likely to flow over the levee and generate potential erosion. One of the first observations is that if the water flows above a levee of height h (also applicable to an embankment), by the time the water reaches the bottom of the dry side of the levee it will have a velocity v, which can be very high. One simple way to evaluate that velocity is to write conservation of energy:

$$mgh = \frac{1}{2}mv^2 \quad or \quad v = \sqrt{2gh} \tag{5.16}$$

Where g is the acceleration resulting from gravity. For example, if the levee is 5 m high (16.4 ft), the velocity v will be approximately 10 m/s (32.8 ft/s). Of course, Equation 5.15 does not take into account the energy lost in friction between the water and the levee surface, but it does indicate that the velocity range is much higher than typically encountered in rivers, where peak velocities range from 3 to 4 m/s (9.84 to 13.12 ft/s). Furthermore, a distinction is made between events such as hurricanes and river floods—the major distinction being that hurricanes may overtop a levee for about 2 hours, while river floods may overtop a levee for 2 days.

A levee-overtopping erosion chart has been developed for these two types of events and is presented in Figure 90, which could be useful for embankments. This chart is based on extensive work by Briaud and his co-workers in the aftermaths of both Hurricane Katrina in New Orleans and the Midwest floods of 2008. By combining Figures 90 and 91 (also Figure 67), one can get a sense of which soils, soil categories, and associated erosion functions are likely to resist overtopping during a 2-hour or 2-day overtopping event. Recall that Categories I to IV on the erosion chart are soils and Categories V and VI are rocks. As shown in the charts, only the most erosion-resistant soils can resist 2 hours of overtopping without protection (Category IV), and no soil can sustain 2 days of overtopping without being totally eroded away. Armoring or vegetation that satisfy strict criteria need to be used to ensure that overtopping can be sustained for longer than 2 hours.

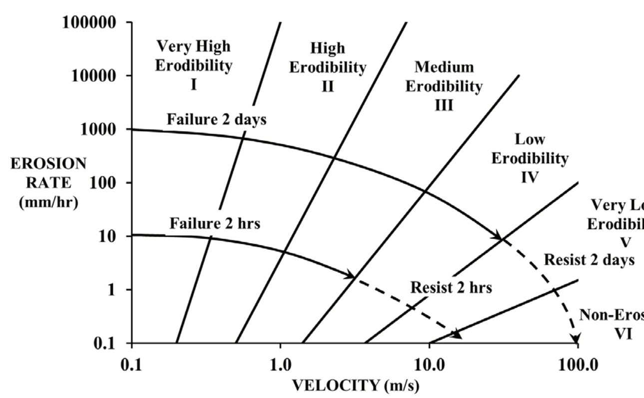

FIGURE 90 Overtopping erosion chart (Briaud 2013).

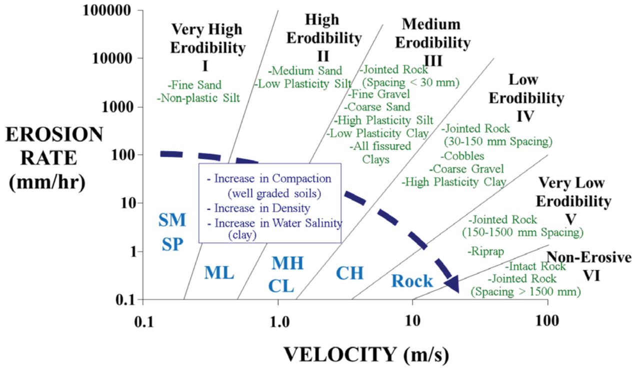

FIGURE 91 Erosion function and soil classification (Briaud 2013).

Vegetation can help significantly to slow down erosion. To be effective, though, this vegetation must satisfy the following minimum requirements:

1. have a mat-like appearance;
2. have a sod-forming root system;
3. be made of perennial grasses;
4. have a dense, consistent coverage; and
5. have a minimum height of 0.3 m during flood season.

Figure 92 shows an embankment that, according to the farmer, was overtopped for 2 days during the June 2008 Midwest Mississippi River flood; it resisted very well because the grass cover met the criteria discussed earlier. Figure 93 shows a weak grass cover on a levee that was overtopped and breached. Figure 94 shows an earth embankment that survived well after 2 hours of overtopping during Hurricane Katrina because the soil was quite erosion resistant (Category IV, Figures 90 and 91 in August 2005) while Figure 95 shows an earth embankment that was totally eroded during the same event (Category I, Figures 90 and 91).

FIGURE 92 Good-quality grass cover to delay erosion during overtopping (Briaud 2013).

FIGURE 93 Poor-quality grass cover unable to delay erosion during overtopping (Briaud 2013).

FIGURE 94 Overtopped earth embankment that survived well after 2 hours of hurricane surge overtopping (Briaud 2008).

FIGURE 95 Overtopped earth embankment totally eroded by 2 hours of hurricane surge overtopping (Briaud 2008).

Tree roots can be considered as reinforcement for the slope of a levee if the tree is on the levee and is alive and healthy. However, if the tree on a levee is uprooted and topples over during a flood, it will create a major hole in the levee. Also, if the tree dies, the disappearance of the roots will leave channels for the water to seep through the levee. On the whole, it is best to not let trees grow on or near levees.

Culvert-Related Problems

Based on the case examples presented and available information, problems associated with culverts are common in flooding events. Erosion is often concentrated at points of discontinuity in a soil mass. This is the case of the interface between a soil embankment and a culvert where internal erosion can be expected. Aggravated damage from erosion at the locations of culverts and overtopping that might have occurred as a result of blocked or undersized culverts are common as revealed by the case examples in chapter three. Based on Iowa's experience with Western Iowa Missouri River Flooding (IHRP TR-638 by Vennapusa et al. 2013), culvert-related problems during flooding include erosion of culvert backfill, separation of culverts, and blockage of water outflow. An example is presented in Figure 96 on Highway 18 (North Dakota). The embankment was overtopped and breached, pavement was damaged, and the culvert was washed out.

FIGURE 96 Pavement damage, washed-out culvert, and embankment breach in 2009 overtopping during spring thaw, Highway 18, North Dakota.

Most DOTs have unique installation methodologies for culverts. The methodologies can include drawings or sections that describe the bedding materials and the placement of the culverts. A number of procedures are included in relevant literature and are currently adopted by some DOTs. Such procedures are related to limiting the scour at culvert inlet and outlet, the allowable headwater behind the culvert, and the culvert bedding. Common scour problems include scour holes and scouring away of the embankment slope materials in the vicinity of the inlet as well as scour at the outlet. This usually occurs as the water velocity increases upon entering the inlet or as it carves its path outside the culvert. Structural problems are related to culvert integrity and proper selection and placement of bedding and fill materials. Available practical solutions are outlined in chapter nine.

According to Bonelli (2013), the presence of a culvert can facilitate the initiation of internal erosion in many ways. Sherard et al. (1972a) and Charles (1997) discuss how a stiff conduit would cause unusual stress distributions when embedded in less-stiff surrounding soil. Also, the drying of surrounding soil could open some cracks that in turn would lead to initiation of piping during flooding. Another factor is that the compaction of the soil in the immediate vicinity of a culvert is often more difficult and less effective. Guidance on relevant most effective practice can be found in FEMA (2006) and guidance on how to assess the likelihood of initiation of erosion around these features is included in Fell et al. (2008).

Pavement Degradation and Failure

Pavement degradation and failures (loss or erosion) are common after and during flooding. Pavement failure comes in different modes, including rafting, edge failure, and post-flood damage (damage that develops after the flood) resulting from the saturation of the pavement and the subgrade. The methodologies to minimize pavement damage during flooding vary among DOTs. Whereas some states specify some measures for this purpose, others do not. This section summarized the failure modes addressed throughout this report. Practical measures adopted to minimize the damage are included in chapter nine.

Rafting

During flooding, pavement failure can occur from rafting, as explained in chapter two. The floodwater can seep into the subgrade and when the uplift pressure becomes large enough, the pavement is carried away by the flow. An example of this phenomenon is presented in Figure 97.

Edge Failure

Another damage mechanism is partial or complete loss of pavement from erosion or loss of the embankment soil support when the upstream slope (seaward slope) or downstream slope (landward slope) is subjected to erosion. Such modes would usually occur in three cases, as explained in chapter two: (1) erosion of the upstream slope (seaward slope) from wave action or toe erosion of the embankment stretching in the vicinity of a stream, (2) erosion of the seaward slope as the water recedes (generally common in coastal environments), and (3) overtopping by surge or waves or the combination of both, which undermines the pavement on the downstream (landward) slope. This mode of failure is common among case examples mentioned in chapter two as well as damage reported by IOWA TR-368. Figure 98 shows a graph of embankment pavement losses prepared by Schneider and Wilson (1980).

FIGURE 97 Pavement rafting damage after Hurricane Isabel in 2003, Kimsey Run Project, West Virginia.

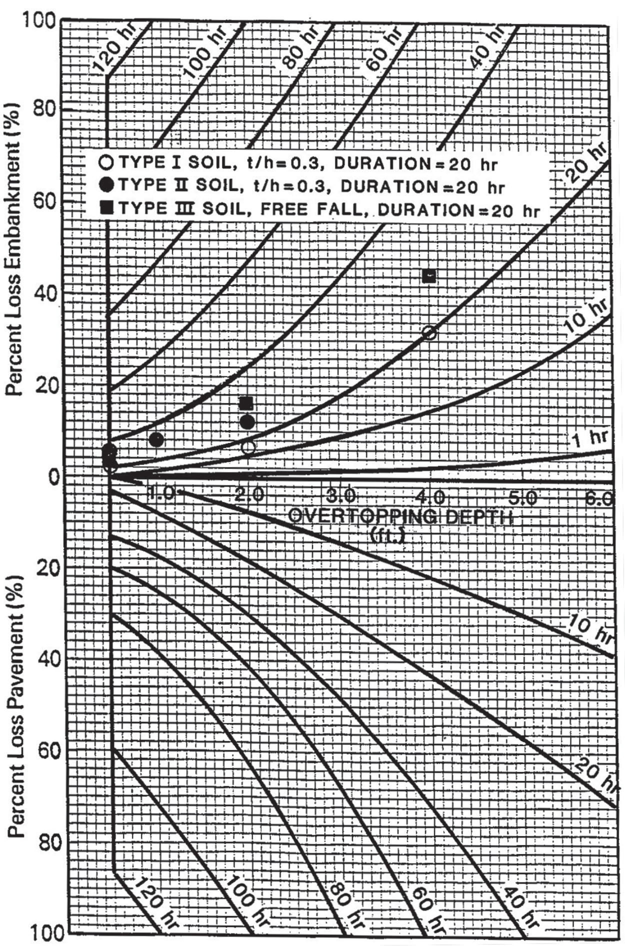

FIGURE 98 Embankment pavement losses (after Schneider and Wilson 1980).

Post-Flood Damage

After a major flood, there may be no visible pavement damage, but the engineer may wonder if some hidden damage exists. Three case examples are presented to address this issue. The discussion in this section is limited to hot mix asphalt (HMA) pavement generally used in roadway highways. Additional information relevant to saturation of the pavement and recovery of strength as a function of time is included in IOWA TR-638 (Vennapusa et al. 2013). Based on the latter, voids were obtained at shallow and deep depths in the aftermath of Western Iowa Missouri River Flooding in 2012. At shallow depths [<150 mm (6 in.)], voids occurred due to erosion of underlying base materials. At deep depths [>150 mm (6 in.)], voids formed due to erosion of subsurface materials.

Case Example 1: TS-1 (Old Mormon Bridge Road), Western Iowa Missouri River Flooding

A research team was assembled right after the Western Iowa River flooding to assess the geo-infrastructure damage, repair, and mitigation strategies. Vennapusa et al. (2013) document their experience with one secondary highway pavement stretch of TS-1 (Old Mormon Bridge Road) that they tested in the course of assessing the pavement condition after flooding.

The pavement was 360-mm (14-in.) thick HMA underlain by 300 mm (12 in.) of thick base over natural subgrade. This stretch was partially submerged for about 2 months during the 2011 flooding event. The goal was to evaluate the pavement performance two times: after the flooding receded and about 8 months later. For this purpose, in situ testing using falling weight deflectometer (FWD), dynamic cone penetrometer, and ground-penetrating radar was carried out in addition to hand-augured soil borings.

The key findings based on the test results are as follows:

- The pavements did not show structural failure, yet granular shoulder erosion occurred close to high water level lines.
- E_{FWD} and E_{SG} were 1.3 to 1.4 times lower in flooded areas than in non-flooded zones. FWD results obtained 6 months after the flood were higher, on average, than the initial values, and those taken about 9 months after the flooding almost matched those of non-flooding areas.
- California Bearing Ratio results for the base layer were almost matching (greater than 50) for flooding and non-flooding sites, unlike the subgrade, which was, on average, about 10 times lower in flooded zones. Measurements taken shortly after and 9 months after these initial measurement were about the same.

Damage basically occurred in the form of voids from the erosion of base and subsurface materials.

Case Example 2: SH-24 North of Washington, Oklahoma/McClain County

Christopher (2007) carried out FWD at this site after the flood. It was concluded that flood durations of 8 to 14 hours were not long enough to cause significant pavement damage. A "slight weakness" was initially observed in the subgrade that was later recovered. The importance of the site geology was further noted in understanding the potential damage to highways. For instance, subgrade made of sandy soils would become severely weak in comparison with clayey soils.

Case Example 3: Pavement Structures Damage/Hurricane Katrina Flooding

Zhang et al. (2008) reported a study carried out by Louisiana Transportation Research Center to assess the impact of Hurricane Katrina on pavements in the region. This study basically concluded that HMA and HMA subgrade layers were significantly weakened by the flooding. It was further noted that more damage was experienced by HMA pavements at lower elevations and by those with less thickness.

Relation Between Overtopping Depth and Pavement Loss

In 1980, FHWA collected data from highway agencies including Schneider and Wilson's work. The data were mainly based on observations of pavement and embankment damage resulting from overtopping. The goal was to develop a relationship between overtopping depth and loss in pavement and embankment. Figure 98 shows the cumulative effects of overtopping over time based on these data (Chen and Anderson 1987).

The embankment test data collected by Chen and Anderson (1987) is also shown on Figure 98 for Type I and Type II embankment soil to be compared with the 20-hour curve. Type I (Clay with low plasticity-CL/USC Unified Soil Classification) shows general agreement with the curve while that of Type II (SM-SC/USC) with freefall condition shows higher erosion rates than the curve.

SUMMARY

This chapter presented geotechnical and geological considerations in minimizing roadway embankment damage from flooding. Geotechnical concepts including critical velocity, erosion function, and the erodibility as a function of soil classification were explained. Available in situ and lab erosion tests were briefly described. Related geological factors include the meandering potential in riverine environments and erosion and deposition in coastal environments. Geotechnical design considerations were then presented for each of the failure modes highlighted in chapter two. Aside from the hydraulic and hydrological aspects presented in chapter four, and the geotechnical and geological aspects presented in this chapter, nontechnical aspects are presented in the next chapter that include legal, regulatory, and funding aspects.

CHAPTER SIX

LEGAL, REGULATORY, AND FUNDING ASPECTS

INTRODUCTION

Design to minimize damage of roadway embankments during flooding is subject to a number of legal, regulatory, and funding limitations. Key issues include deciding how to design the embankment and determining the available funding options. Design considerations include the purpose of the structure and the design freeboard to mitigate overtopping potential and minimize the foreseen damages. Federal disaster funding reimbursement programs are FHWA ER and FEMA public assistance. Other considerations and constraints also play a role in the decision-making process for roadway embankment design. Such constraints include right-of-way limitations, stream and wetland impacts that could often occur in floodplains, environmental considerations, permitting, feasibility and time considerations, and prioritization of projects. Because limited information was gathered about these considerations, this section will only discuss the purpose of the structure, the freeboard issue, and the federal funding disaster reimbursement programs.

DESIGNING EMBANKMENTS AS LEVEES

As explained in chapter one, roadway embankments differ in purpose from flood control structures such as levees. After most of the National Highway System was built, FEMA carried out a program to identify areas threatened by flood damages through flood insurance studies that are a part of the National Flood Insurance Program. Some roadway embankments lie within the high-risk areas and were mistakenly classified as levees.

In this regard, FHWA issued a memo (dated September 10, 2008) in response to incorrect designations of some roadway embankments as levees and to levee certification initiatives. The memo discourages certifying embankments as levees and advises against any assumptions that embankments offer any degree of protection against flooding. As stated by some of the participating states in the survey, DOTs do not design embankments as flood control structures. This is generally discouraged but allowed in very few cases. In such cases, USACE is the agency responsible for the levee certification. Designing to protect against flooding, however, has become an increasing concern. The available options are either to increase the freeboard to avoid overtopping or to provide adequate design with suitable measures to protect the embankment during extreme events. These options are further discussed in the next section.

FREEBOARD ISSUE

The freeboard is a form of safety factor that is applied as feet above a certain level. This freeboard provides a margin of safety against unforeseen issues that include a flood larger than the design flood, an unanticipated failure in a culvert, a wave higher than the design wave, and so on. Freeboard considerations as applied to roadway embankments are subject to state-by-state requirements.

So why is increasing the freeboard height not adopted as an alternative solution to prevent overtopping, to avoid costly protection measures and to minimize damage resulting from other factors caused by high headwaters?

The reason is that roads subject to flooding action generally lie within the floodplain. The floodplain is subject to regulations imposed by a number of agencies that include FEMA, the state's National Flood Insurance Program Coordinating Agency, local watershed boards, and related authorities. Depending on the type and location of the roadway embankment, a number of requirements need to be considered to allow for an increase in the freeboard. For example, if the roadway embankment lies within the regulatory floodway, no development is allowed unless the developer can prove that this development would not lead to an increase in the flood elevation at any location during the 100-year flood. Such processes are time consuming and not fruitful. As a result, different alternatives are visited.

It is worth noting that, in some cases, it may be desirable to have a fuse plug where the embankment is lower than the rest of the roadway over a short length. This ensures that overtopping will occur at that location, which allows the fail to occur in a controlled fashion. Also, in coastal cases, the roadway is designed to be overtopped on the assumption that sand will cover the roadway during overtopping, thereby protecting it from excessive damage. Such practices should be assessed on a case-by-case basis.

FUNDING OPTIONS

Failures among roadway embankments resulting from flooding are commonly encountered, and they impose challenging situations and burdensome costs. Two federal emergency reimbursement programs—FHWA Emergency Relief (ER) and FEMA Public Assistance (FEMA PA)—were developed to supplement state and local resources to address the financial challenges imposed by extraordinary conditions such as natural disasters. FHWA's most recent ER program manual is the *Emergency Relief Manual* (Federal-Aid Highways) released May 31, 2013. FEMA's most recent PA handbook is FEMA P-323, the *Public Assistance Applicant Handbook* (2010). The programs feature significant differences and are not intended to fully reimburse state DOTs. Nakanishi and Auza (2015) provide a synthesis on FEMA PA and FHWA ER reimbursements to state DOTs. The synthesis presents a valuable resource on relevant current practice and latest literature. It includes a description of the programs and interviews with 10 state DOT personnel to identify effective practices based on case examples. The following paragraphs state the aim of each of these programs and highlight the differences in scope and funding.

The FHWA ER program is administered by FHWA and aims to "aid States in repairing road facilities which have suffered widespread serious damage resulting from a natural disaster over a wide area or serious damage from a catastrophic failure" [Code of Federal Regulations (CFR), Title 23, § 668.105, "Policy"]. The facilities eligible for this program are federal-aid highways; federal-aid highways are all public roads including bridges that are not classified as local or rural minor collectors (or minor collectors located in rural areas) [U.S.C., Title 23, § 101(a) (5)].

FHWA ER's goal has been to fix damage resulting from catastrophic events and not to fix "preexisting, non-disaster related deficiencies" (CFR, Title 23, § 668.105, "Policy").

Based on the *Emergency Relief Manual* (FHA 2013), the purpose of the ER funds is to provide reimbursement for the repair or reconstruction costs of the federal-aid facilities. Title 23 Code of Federal Regulations (23 CFR) Part 668, Subpart A states that an event generally must have caused at least $700,000 (federal share) in eligible damage for the event to be eligible for ER funding. If the disaster damage is less than this value, it is generally considered to be heavy maintenance or routine emergency repair. The *Emergency Relief Manual* identifies two categories of emergency relief: emergency repairs and permanent repairs. Emergency repairs are those undertaken during or immediately after a disaster to restore essential traffic, to minimize the extent of damage, or to protect the remaining facilities. Permanent repairs are those undertaken after the occurrence of a disaster to restore the highway to its pre-disaster condition. The ER program allows for betterments in permanent repairs, if justified. Betterments are added protective features, such as the rebuilding of roadways at a higher elevation or the lengthening of bridges, or changes that modify the function or character of a highway facility from what existed before the disaster or catastrophic failure, such as additional lanes or added access control (CFR, Title 23, § 668.103, "Definitions"). Betterments can be justified for ER funding by comparing the projected cost to the ER program from potential recurring damage over the design life for the basic repair to the cost of the betterment (FHWA 2013).

TABLE 13
ELEMENTS OF FEMA PA AND FHWA ER PROGRAMS[1]

Program Elements	FEMA Public Assistance	FHWA Emergency Relief
Facility	Non federal-aid facilities, except for debris	Roads and bridges on federal-aid highways
Cause	Major disaster or emergency	"Natural disaster, catastrophic failure due to external cause"
Cause-Fire	If there is an uncontrolled forest, woodland, or grassland fire, consider seeking FEMA Fire Management Assistance Grant funding	
Declaration required	Presidential declaration	Presidential declaration or governor's declaration/proclamation
Declaration impact indicators/minimum thresholds	Impact indicators: Statewide per capita impact indicator, $1.39; countywide per capita impact indicator, $3.50 (cf. 44 CFR 206.48; impact indicators updated annually in the *Federal Register*)	Minimum thresholds for federal share: $700,000 statewide [23 CFR 668, Subpart A; exceptions are listed in 23 CFR 668.105(j)]
Scope	N/A	Wide area affected (e.g., multiple counties)
Project size	Differentiates between small and large projects	Does not differentiate between small and large projects
Applicant	Subgrantees: state and local governments, tribes, eligible Provincial Nominee Program	State DOT
Emergency repair/work (federal share)	Minimum of 75% (p. 2, FEMA P-323, 2010)	100% (p. 13, *ER Manual*, 2013)
Permanent restoration (federal share)	Minimum of 75% (p. 2, FEMA P-323, 2010)	Minimum of 90% for Interstate, 80% for other federal-aid highways (p. 52, *ER Manual*, 2013)
Project form	Project worksheet usually prepared by FEMA	DDIR/DAF completed by state DOT
Documentation retention	Minimum of 3 years from the date of the final status report	Minimum of 3 years after FHWA's closeout of final voucher

N/A = not applicable
Note: Minimum per project for FEMA PA projects has been increased from $1,000 to $3,000. For FEMA PA projects, the president may increase the federal share and occasionally has done so for limited emergency work if severe damage is noted (e.g., Hurricane Sandy in 2012 and the Gulf Coast disasters of 2005). Further, there is a federal cost share increase under standard procedures for alternate projects (FEMA Headquarters Public Assistance Division Staff, April 30, 2014). Note that the state determines how the nonfederal share is assigned to each of its subgrantees.
[1] After Nakanishi and Auza (2015).

The FEMA PA program has a larger scope than that of the FHWA ER program. The applicant to the FEMA PA program could be a state agency, local government, Indian tribe, authorized tribal organization, Alaska Native village or organization, or one of certain private nonprofit organizations that submit a request for disaster assistance under the presidentially declared major disaster or emergency. The program's mission is "to support our citizens and first responders to ensure that as a nation we work together to build, sustain, and improve our capability to prepare for, protect against, respond to, recover from, and mitigate all hazards" (FEMA 2014). Unlike FHWA ER, which is cost-reimbursable, FEMA PA essentially is grant funding. Based on the *Public Assistance Applicant Handbook* (FEMA 2010), the federal government shares the cost with the applicant (usually 75% federal and 25% nonfederal) according to the FEMA–State Agreement (a formal legal document stating the understandings, commitments, and binding conditions) for the major disaster or emergency. When conditions warrant and if authorized by the president, 100% federal funding may even be available for a limited period. Similar to FHWA ER, FEMA PA also differentiated between emergency and permanent repair works. Unlike the FHWA ER program, however, FEMA PA doesn't allow for betterments and requires an "in-kind" replacement.

Table 14 presents a comparison between the elements of the two programs (Nakanishi and Auza 2015). It mainly covers the facilities eligible for the aid, the cause of damage, the declaration required for eligibility to the funds, the scope covered by the program, the project scale, the eligible applicants, the emergency work, the current restoration, the project form, and the documentation retention. As shown through the comparison, the scope of FEMA PA is broader than that of FHWA ER in terms of the range of projects and applicants.

PREPAREDNESS

An important part of dealing with flooding consequences is preparedness. Based on the synthesis carried out by

TABLE 14
ELIGIBILITY CRITERIA FOR FEMA PA AND FHWA ER PROGRAMS[1]

Measure Needed	FEMA PA	FHWA ER
Emergency work/repair	Categories A and B: performed "before, during, and following a disaster to save lives, protect public health and safety, or eliminate immediate threat of significant damage to improved public and private property through cost effective measures" (FEMA 2010, p. 14)	Performed during or immediately after a disaster to • Restore essential traffic, • Minimize the extent of damage, or • Protect the remaining facilities (FHA 2013, p. 3).
Debris removal (Category A)	Debris removal from non-federal-aid highways. Given a presidential declaration and a FEMA determination, removal of debris from federal-aid highways may be eligible under sections 403, 407, or 502 of the Stafford Act. Requirements in 44 CFR 206.224 apply (FHA 2013, p. 10).	Debris removal under certain circumstances. Disaster-related debris removal that is eligible for FEMA funding is not eligible for FHWA ER funds (FHA 2013, p. 10).
Emergency protective measures (Category B)	Examples include: • Emergency evacuations • Protection for an eligible facility • Security in the disaster area • Warning of risks and hazards (FEMA 2010, p. 14)	N/A
Permanent work/repair	• Must repair, restore, or replace disaster-damaged facilities in accordance with regulations • Must restore to predisaster design capacity, and function in accordance with applicable codes and standards (FEMA 2010, p. 15) • Must be required as a result of the disaster (FEMA 2010, p. 6) • May include cost-effective hazard mitigation measures (FEMA 2010, p. 23) • Project improvements or alternative projects may be proposed	Undertaken after a disaster to restore the highway to a comparable facility. Comparable facility is defined as "a facility that meets the current geometric and construction standards required for the types and volumes of traffic that the facility will carry over its design life" (FHA 2013, p. 2). Note that "[f]features that will improve the resilience of repaired federal aid highways should be considered and evaluated consistent with risk, cost effectiveness and regulatory conditions" (FHA 2013, p. 60).
Eligible cause	Presidentially declared disaster or emergency (FEMA 2010, p. 1)	Natural disaster or catastrophic failure (FHA 2013, p. 1).
Eligible elements	Non-federal-aid highways (except for debris removal) and other facilities	Elements within the cross section of a federal-aid highway (FHA 2013, pp. 1 and 3).

N/A= Not applicable
[1] After Nakanishi and Auza (2015)

Nakanishi and Auza (2015), state DOTs are generally knowledgeable of FHWA and federal aid processes and related procedures. However, they tend to be less familiar with FEMA's processes and requirements. State DOTs also consider additional training on both funding programs to be desirable for local public agencies. Additionally, to maximize the eligible reimbursement, state DOTs have to meet applicable design and construction deadlines specified by funding agencies.

SPECIAL CONSIDERATIONS

Based on Nakanishi and Auza (2015), special considerations apply to both programs that could affect eligibility and the scope of work. Such considerations include the following: environmental protection, insurance, hazard mitigation, historic preservation, cultural resources, and floodplain management.

Compliance with all environmental protection laws and regulations is required and includes the following:

- National Environmental Policy Act (NEPA)
- Endangered Species Act
- Clean Water Act
- Clean Air Act.

Although an FHWA ER program project shall comply with the NEPA of 1969, "emergency repairs and in-kind replacements usually receive categorical exclusions. However, an environmental evaluation can be required though for FHWA ER project that includes betterments (Nakanishi and Auza 2015).

SUMMARY

In addition to hydraulic, hydrological, and geotechnical considerations in roadway embankment design, other considerations interfere in the decision-making process. These considerations include right-of-way constraints, stream and wetland impacts that could often occur in floodplains, environmental considerations, permitting, feasibility and time considerations, prioritization of projects, the purpose of the embankment, the freeboard issue, and the federal funding disaster reimbursement programs. This section elaborated on the purpose of latter three factors based on information gathered through the survey and interviews. The next chapter presents concepts in probability and risk that would further affect the decision-making process.

CHAPTER SEVEN

PROBABILITY AND RISK

DETERMINISTIC, PROBABILISTIC, AND RISK-BASED APPROACHES

Engineering design involves calculations and decisions that are associated with uncertainty. Three approaches can be selected:

1. The deterministic approach, where best estimates of parameter values are selected, typically the mean value, and a global factor of safety is applied to the result to minimize the possibility of failure.

2. The probabilistic approach, where in addition to making calculations using the mean values of the parameters, the uncertainty associated with the answer is quantified by calculating the probability of failure, usually through the use of standard deviations. Then an acceptable probability of failure is selected and the design requirements are back-calculated to satisfy this very low target probability of failure.

3. The risk approach, where in addition to calculating the probability of failure, the value of the consequence is introduced in calculating the risk expressed as the product of the probability of failure times the value of the consequence. Then an acceptable risk value is selected and the design requirements are back-calculated to satisfy this very low target risk.

Fifty years ago, engineering design was dominated by the deterministic approach; today, the probabilistic approach is well on its way to becoming the dominant approach. In the future, the risk-based approach is likely to be chosen by the profession. The advantages and drawbacks of each approach are discussed next.

Further reading on the subject topic should include the following:

- Baecher and Christian (2003), *Reliability and Statistics in Geotechnical Engineering*
- Fenton and Griffiths (2008), *Risk Assessment in Geotechnical Engineering*
- Phoon and Ching (2015), *Risk and Reliability in Geotechnical Engineering.*

ADVANTAGES AND DRAWBACKS OF THE DETERMINISTIC APPROACH

An advantage of the deterministic approach is that the calculations correspond well with what happens in reality. For example, the best estimate of the water velocity in the river during a big flood is used and the engineer gets a sense of what is happening near the embankment. At the same time, this estimate is associated with uncertainty, which is absorbed in a single factor of safety; this factor of safety is the same regardless of the level of uncertainty. Consider Figure 99, which shows the probability density function for two cases. In the case of curve A, the soil properties of the embankment are relatively well known, the soil is relatively uniform, and the standard deviation is small. In the case of curve B, the soil properties of the embankment are not as well known, the soil is more heterogeneous, and the standard deviation is larger. By definition, the probability of failure is the area under the curve to the left of the axis corresponding to a factor of safety equal to 1. As the figure illustrates, this area is much larger in the case of curve B than in the case of curve A. Yet the mean factor of safety is the same at 1.5. It does not make sense to use the same factor of safety in both cases. This is why the probabilistic approach was developed.

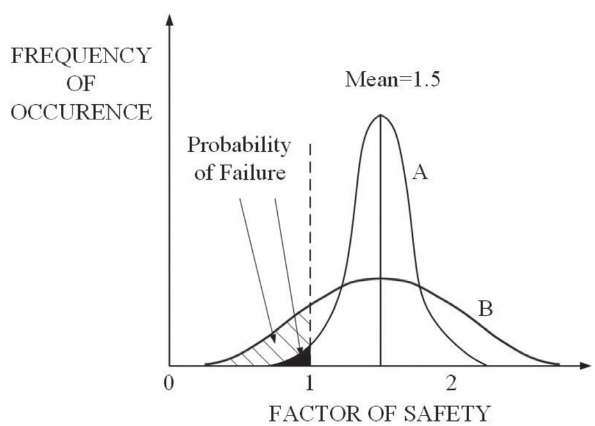

FIGURE 99 Difference in probability of failure for the same factor of safety.

ADVANTAGES AND DRAWBACKS OF THE PROBABILISTIC APPROACH

As pointed out earlier, the probability of failure is the area under the curve to the left of the axis corresponding to a factor of safety equal to 1. In the probabilistic approach, the first step is to determine the probability density function for the soil in the embankment, for example. This is often reduced to evaluating the mean and the standard deviation for the data available. Then calculations include determining the factor of safety, which corresponds to the mean of the distribution plus the probability of failure. The probability of failure is added information that helps the engineer to incorporate the variability of the input data into the engineering decision. Now the engineer can select an allowable target probability of failure and back-calculate the factor of safety that will satisfy not only the mean value but also the allowable probability of failure. This is, for example, what is practiced with the use of the Load and Resistance Factor Design, where an allowable target probability of failure of the order of 0.001 (1 chance of failure in 1,000) is considered acceptable for most structural components. Table 15 shows data on probability of death for humans to be compared with the previously mentioned target value in civil engineering of 0.001. This target probability of failure is a very useful concept, but it does not make sense to design a warehouse in the middle of nowhere for the same probability of failure as a nuclear power plant next to a megacity. The value of the consequence, should failure occur, is important. This is what prompted engineers to think in terms of risk instead of only probability of failure.

ADVANTAGES AND DRAWBACKS OF THE RISK APPROACH

The risk R is defined as

$$R = PoF \times C \tag{7.1}$$

Where PoF is the probability of failure and C the value of the consequence. This consequence can be evaluated in terms of the number of fatalities F or in terms of the number of dollars lost D.

$$R(fatalities) = PoF \times F \tag{7.2}$$

$$R(dollars\ lost) = PoF \times D \tag{7.3}$$

The probability is usually an annual probability of failure; the risk R is therefore in units of fatalities per year or dollars lost per year. Often Equation 7.1 is more properly written as

$$R = T \times V \times C \tag{7.4}$$

Where T is the threat, V the vulnerability, and C the value of the consequence. As can be seen in this case, the probability of failure is split into two components. The threat is the probability that a certain event will occur (big flood or big earthquake), whereas the vulnerability is the probability that failure will occur if the event occurs. Vulnerability is the part of the system where one has the most control. Fragility curves (Figure 100) link the probability of failure to the severity of the threat; they quantify the vulnerability of the system through the function V.

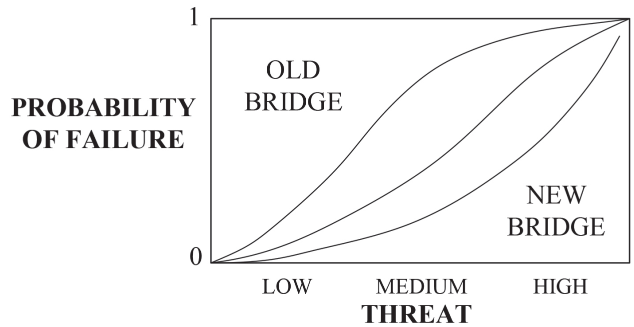

FIGURE 100 Fragility curves.

FIGURE 100 Fragility curves.

TABLE 15
PROBABILITY OF DEATH FOR HUMANS (BRIAUD 2013)

Activity	Probability of Death
Heart disease	0.25
Cancer	0.23
Stroke	0.036
Car accident	0.012
Suicide	0.009
Fire	0.0009
Airplane	0.0002
Bicycle	0.0002
Lightning	0.00001
Earthquake	0.000009
Flood	0.000007

Note that it is not possible to design a structure (for example, an embankment) that has zero risk associated with its engineering life. This is because any calculation is associated with some uncertainty; that the engineering profession's knowledge, though having made great strides, is still incomplete in many respects; that human beings are not error free; and that the engineer designs the structure for conditions that do not include extremely unlikely events such as a falling satellite hitting the structure at the same time as an earthquake, a hurricane, and a 500-year flood during rush hour.

The choice of an acceptable risk is difficult because so many factors enter into the decision including the replacement cost and the number of lives at risk. The choice of an acceptable risk also involves other disciplines such as philosophy, politics, and social sciences. One of the very difficult steps required in estimating an acceptable risk is deciding what price to put on human life. It is not uncommon to use a number such as $1 million, because that is an average life insurance value for many people.

Figure 101 shows the annual risk associated with various engineering activities and in everyday life. The annual probability of failure (*PoF*) is on the vertical axis, and there are two scales on the horizontal axis: lives lost or fatalities per year (*F*) and dollars lost per year (*D*). Because the two do not necessarily correspond, the activities are shown as bubbles rather than precise points on the graphs. Because the risk is the product of the probability times the value of the consequence, two risk values can be defined: *R* (fatalities) = *PoF* × *F* and *R* (dollars lost) = *PoF* × *D*. Therefore, the annual risk is constant on diagonals in Figure 101. The dotted, dashed, and solid lines correspond to a high, medium, and low annual risk. The numbers are shown in Table 16. These data indicate that, in the United States, 0.001 fatalities per year and $1,000 per year may be acceptable target risk values.

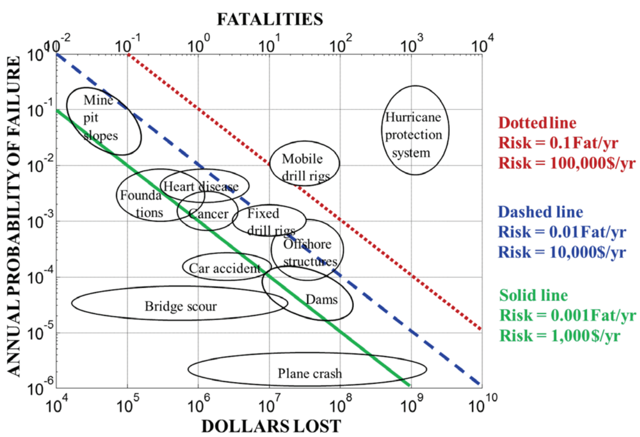

FIGURE 101 Risk associated with various engineering and human activities (Briaud 2013).

TABLE 16
ANNUAL RISKS FOR THE UNITED STATES (RISK = *POF* X VALUE OF THE CONSEQUENCE)

Annual Risk Level	Fatalities/Year in the United States	Dollars Lost/Year in the United States
Low	0.001	1,000
Medium	0.01	10,000
High	0.1	100,000

Risk = *PoF*x value of the consequence.

The advantage of the risk approach is that it is the most effective decision tool available today. The drawback is that it requires complex calculations that require a fair amount of information not always available to the engineer.

DESIGN FLOOD AND ASSOCIATED PROBABILITY OF EXCEEDANCE

One important engineering decision that affects risk is the choice of the design flood. This design flood can be the 10-year flood, 50-year flood, 100-year flood, 500-year flood, or other timespan. The probability of exceedance associated with this choice can be calculated as follows. As explained previously, the probability of exceeding the 100-year flood is 0.01 or 1 chance in 100 in any one year. This means that each year the probability is the same, regardless of whether a 100-year flood has occurred in the previous year. However, as can be imagined, if an embankment is going to be designed for, say, 75 years, as is the case for the design life of a bridge, then the probability that the 100-year flood will occur or be exceeded during those 75 years is higher than during any one year. The probability of exceedance in 75 years PoE_{75} is given by

$$PoE_Y = 1 - (1 - PoE_1)^Y \tag{7.5}$$

Where PoE_Y is the probability of exceedance in Y years, and PoE_1 is the probability of exceedance in any one year. Let us consider the 100-year flood for example together with the 75-year typical design life. Equation 7.6 gives

$$PoE_Y = 1 - (1 - 0.01)^{75} = 0.53 \tag{7.6}$$

Therefore, the probability that the 100-year flood will occur or be exceeded during a 75-year design life is over 50%. For the 500-year flood, this probability becomes 0.14 or about 1 chance in 7 that the 500-year flood will occur or be exceeded during the design life of the embankment. If we go back to the acceptable probability of exceedance of 0.001 typical in civil engineering, a back calculation using Equation 7.7 gives

$$PoE_Y = 0.001 = 1 - (1 - PoE_1)^{75} \quad or \quad PoE_1 = \frac{1}{75151} \tag{7.7}$$

In other words, the 75,000-year flood corresponds to a *PoE* of 0.001 for a design life of 75 years. This appears extreme, but remember that this only requires to consider an 83% increase in velocity compared with the velocity for the 100-year flood (see the chapter four section "Impact of Recurrence Interval on Velocity and Water Depth").

SUMMARY

This chapter explained the deterministic, probabilistic, and risk approaches that are adopted to deal with the factor of uncertainty presented in the design. The advantages and drawbacks of each of the approaches were discussed. Because the selection of a design flood is a very important decision in the design process, the design flood and the associated probability of exceedance are also explained. The next chapter presents a summary of the survey results obtained from the DOT engineers.

CHAPTER EIGHT

SUMMARY OF SURVEY RESULTS

INTRODUCTION

This chapter presents a summary of the results received from the DOT engineers. Of the 50 total responses that were received, 32 made major contributions to this study. Additionally, two answers were retrieved from FHWA Colorado and FHWA Washington. In addition to the survey responses, many telephone interviews were conducted. Figure 102 shows the state DOTs that made major contributions to the synthesis.

The survey questionnaire is attached in Appendix A. The results are documented under the following categories: case examples, documents used in current practice, modes of failure, geological considerations, hydraulic and hydrologic considerations, geotechnical considerations, construction considerations, protection, maintenance and control techniques, probability and risk, decision-making process and funding, and current and future research areas. The relevant survey question is presented at the beginning of each section, followed by a summary of the responses.

CASE EXAMPLES

Q1. Please provide up to three success case examples of a highway embankment subjected to flooding and why you think it was successful.

Q2. Please provide up to three failure case examples of a highway embankment subjected to flooding and why you think it failed.

A total of 41 case examples were gathered through the survey. Fourteen of these studies were included in chapter three. The rest are shown in Tables 17 and 18. In Table 17, successful case examples are reported in which the design could withstand the flooding event.

Table 18 present failure case examples in which the embankment was damaged in the flooding event. The cause and extent of damage are included for each case as applicable.

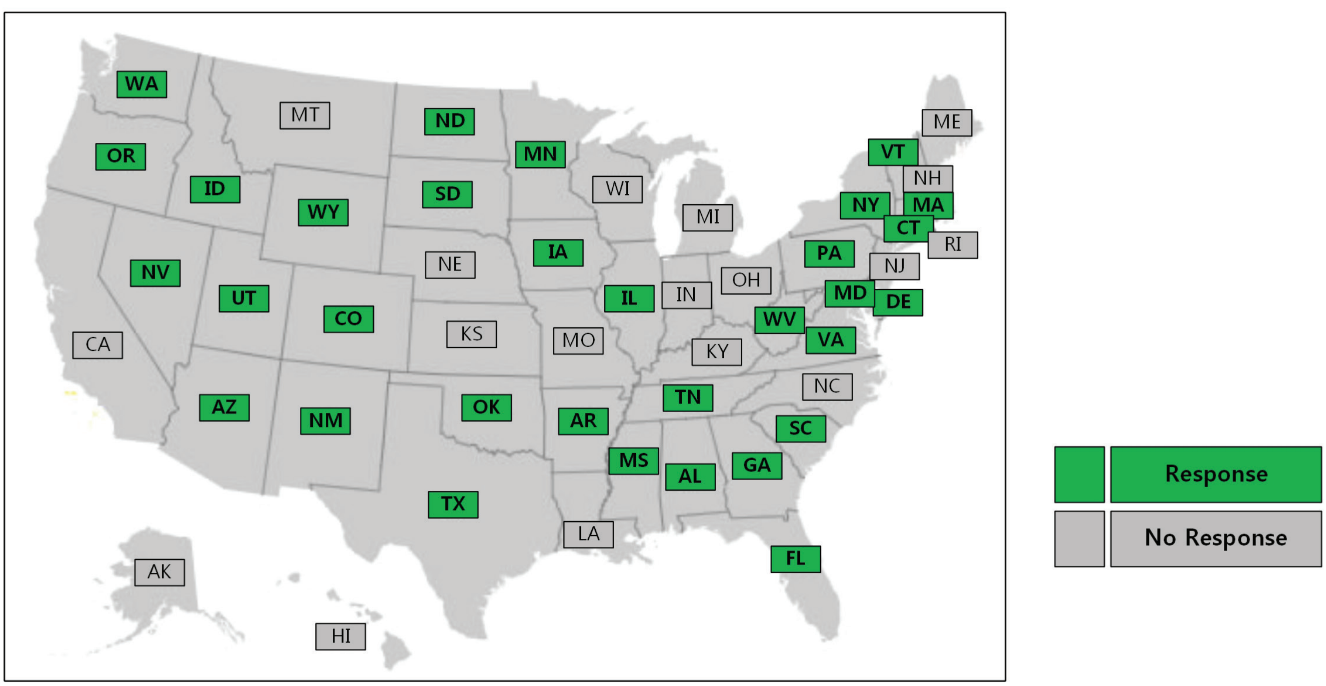

FIGURE 102 The participating states in the synthesis in green.

TABLE 17
SUCCESS CASE STUDIES GATHERED THROUGH THE SURVEY

Agency	Project	Comments
Arizona DOT	Ideal Draw near Wilcox	The wingwall and the box culvert were undermined.
	San Pedro on I-10 near Benson	This is an example of bridge deck and roadway drainage effect during flooding.
	Laguna Wash on 163 near Kayenta	The roadway embankment was affected by meandering of the river.
Iowa DOT	I-80 over Cedar River in Cedar County	Paved shoulder mitigated damages to pavement and the damage was observed in the roadway embankment.
	US 65 over Des Moines River in Polk County	Minimal damage was caused due to flat embankment slope.
Nevada DOT	Wilson Canyon	This is a clearing and dredging project that placed spur dikes to protect roadway.
	Rainbow Canyon Reconstruction	Overtopping areas constructed and have since been flooded, but remain intact.
Oklahoma DOT	Tropical Storm Erin 1 Blaine Co SH 33 over creek 10.8 mi east of US 281 in Watonga	This was embankment failure due to contraction scour from Tropical Storm Erin. Thankfully the bridge did not fail and we were able to reopen quickly.
	Grady Co US 81 over Buggy Creek south of Minco	This bridge had abutment wash out during tropical storm Erin. Roadway embankment was ok because highway overtopped at the fuseplug. The bridge was reopened quickly.
	Flooding after Tropical Storm ErinCaddo Co SH 146 over Punjo Creek and Tribs	Roadway overtopped and had to be cleared of debris and silt after debris clogged RCB. A grader was used to clear roadway of silt and tree limbs.
Pennsylvania DOT	Spruce Street Retaining Wall	This is both a failure and hopefully a successful case when construction is complete. The retaining wall protecting a rail line was severely undercut and is being repaired.
Texas DOT	FM 20 at San Marcos River	The channel was armored approximately 4 years ago and is performing well.
	Loop 150 and Colorado River	This is an example of slope repair and bank toe protection
	IH 10 at North Llano River Junction Texas	This is an example of a steep bank armored with gabion faced MSE wall.
Utah DOT	Weber River, I-84 Stabilized roadway embankment	The Weber River runs parallel with Interstate-84 within Weber Canyon. Annually during spring runoff the river experiences high flow events. The reinforced embankments along this stretch of the interstate have remained stable.

TABLE 18
FAILURE CASE STUDIES GATHERED THROUGH THE SURVEY

Agency	Project	Comments	Damage Extent	Cause of Damage
AZDOT-F	Ideal Draw near Wilcox	The wingwall and the box culvert were undermined.	_	Softening due to saturation/Underseepage/Other (s): not having concrete apron between wingwalls
	San Pedro on I-10 near Benson	This is an example of bridge deck and roadway drainage effect during flooding.	Affecting traffic	Softening due to saturation/Other(s): water in traveling lane
	Laguna wash on 163 near Kayenta	This is a case of settlement of bridge barrier.	Partial damage-pavement damage	Softening due to saturation
FHWA	Middle Fork Snoqualmie River Road	Flow spread across debris fan crossed road in several locations and induced road surface and emabankment erosion. One low water crossing was undermined.	Partial erosion	Overtopping
Iowa DOT	IA 150 over Cedar River near Vinton	Head differential caused major breach and undermining of pavement.	Full breach	Overtopping
	US 6 over Cedar River in Muscatine County	Minimal damage was caused due to flat embankment slope.	Partial erosion - pavement damage	Overtopping/Other(s): loss of shoulder and undermining of pavement/Internal erosion

Agency	Project	Comments	Damage Extent	Cause of Damage
North Dakota DOT	ND 18 from Neche to Canadian border	During spring thaw after particularly heavy winter snow seasons in southern Manitoba/northeastern North Dakota, the Pembina River floods out of its banks. ND Highway 18 has been overtopped numerous times, with particularly serious damage in 2009. Washed out culverts and loss of pavement and soil in areas above these culverts was the primary damage.	Full breach-pavement damage	Overtopping
New York State DOT	Route 103 over Mohawk River	NY Rt. 103 crosses the Mohawk River on top of the Erie Canal's dam and lock structure at Lock 9. Just north of the bridge Route 103 continues across the north side of river some 400 ft. to shore. Back-to-back storms set up this incident in 2011. Hurricane Irene was approximately the Q100 for this location on the Mohawk (127,000 cfs). It was followed 8 days later by Tropical Storm Lee, which was close to the Q50 (116,000 cfs). Both floods resulted in tremedous amounts of trees and other floating debris. When the floating debris hit the dam, it was caught and blocked flow. The river rose up and decided to carve itself a complete new channel north of the lock structure. DOT's bridge survived fine (abutment and pier perched on the piles) but the 40 ft high approach embankment was overtopped and washed away in hours. Subsequently rebuilt with armoring and controlled overflow section.	Full breach	Overtopping
Oklahoma DOT	1 Blaine Co SH 33 over creek 10.8 mi east of US 281 in Watonga	This was embankment failure due to contraction scour from Tropical Storm Erin. Thankfully the bridge did not fail and we were able to reopen quickly.	Full breach-pavement damage	Overtopping
	Alfalfa Co17668-04 SH 8 from Major CL N 4.0 mi	April 27, 2009, flooding caused wash out of 8 cell RCB and roadway fill.	Full breach-pavement damage	Overtopping Underseepage
Oregon DOT	Little South Fork Hunter Creek	This is a case of bridge abutment washout. The advantage is this bridge was designed for a washout condition and needed the approach to be rebuilt, easier than reconstructing the bridge.	Full breach	Bridge abutment scour
	Mt. Hood Highway, OR 35, Nov. 2006	Debris flows from glacial outburst clogged up culverts and bridges along a stretch of road and cut through the roadway at others.	Full breach-partial erosion-pavement damage	Overtopping
Pennsylvania DOT	Spruce Street Retaining Wall	–	Severe erosion underneath the wall affecting the rail line	Internal erosion/Softening due to saturation underseepage
Texas DOT	FM 787 at Trinity River	This is the case of ongoing issues with bank erosion and instabilty.	Pavement damage	Internal erosion/Rapid drawdown
	US 79 at Tinity River Relief near Palestine Texas	The toe erosion of the bank has resulted in slope instability.	Partial erosion	Failure due to sliding on foundations
Utah DOT	Cannonville Bridge Embankment Failure	The roadway embankment on the east side of an existing bridge south of Cannonville, Utah, failed due to high flows in the Paria River. The east-side stream bank upstream of the bridge was lined with rock riprap, however high flows eroded the natural ground behind the rock riprap causing the stream bank to erode. This erosion washed the easterly bridge approach.	Full breach-partial erosion-pavement damage	Softening due to saturation/Erosion occuring behind riprap

DOCUMENTS USED IN CURRENT PRACTICE

Q3. Do you have hyperlinks, hard copies, and/or soft copies of the following documents? Please check the documents provided and fill in the reference.

A list of the documents gathered through the survey is included in the Bibliography. The documents fall under the following topics: design documents currently used by the design agency, design criteria, documents considered helpful in design, specifications, sources of hydraulic and hydrologic equations, sources of geologic and geotechnical information, documents used in assessing the embankment condition for resistance to forces generated during flooding, documents used for redesign, maintenance and repair of damaged embankments, documents that provide guidance on legal and regulatory factors included in the design, maintenance and repair of embankments, and other documents gathered through e-mails or that were not listed under the aforementioned categories.

MODES OF FAILURE

Q4. Please specify/rank the highway embankment failure modes that your agency has experienced.

A range of failure modes is witnessed from flooding, including overtopping, saturation, underseepage, through-seepage, wave erosion, and sliding on foundations. The occurrence of a certain mode and the extent of damage are dependent on site-specific conditions. Figure 103 shows the common failure modes among the participating states. The DOT engineers were asked to rank the failure modes from least common (1) to most common (5). Based on the results shown in Figure 103, overtopping is classified as the most common failure mode in five states, followed by softening owing to saturation in three states and underseepage in one state. The second most common mode is overtopping in two states, followed by softening from saturation, underseepage, through-seepage, and sliding on foundations in one state each. The moderately common mode (rank 3) is overtopping in six states followed by saturation and underseepage in five states each, sliding on foundations in four states, and through-seepage and wave erosion in three states each.

Additional notes were provided by the participants related to saturation, failing culverts, and toe erosion due floodwaters running in road ditches. Heavy precipitation can cause saturation. Saturation can lead to slope failures or sliding on foundations. Culvert-related issues include scour or erosion around culverts, separation between the culvert and the surrounding soil, and under-designed or plugged culverts by debris. Toe erosion could occur in the presence of ditches running alongside the toe, in the event water overflows from the ditches. Another cause is the lateral migration of meandering streams during floods eroding the toe and leading to instability and sliding on foundations.

GEOLOGICAL CONSIDERATIONS

Q5. What are the geologic and geomorphic factors that should be addressed in the design of highway embankments subjected to flooding? Why?

In this section, respondents were asked to rank the geologic and geomorphologic factors from the most important (5) to the least important (1). Erosion and deposition and meandering potential ranked as the most important in eight states, followed by channel dimensions and topography in seven states, and floodplain and basin roughness in four and two states, respectively (Figure 104). The second most important factors were considered to be floodplain in 11 states and meandering potential and topography in nine states. It was further noted by a couple of participants that the importance of these factors varies depending on the case considered, whereas others answered based on their experience. Other factors considered included the history of head cutting and the available materials that could be used for the purpose of the embankment fill.

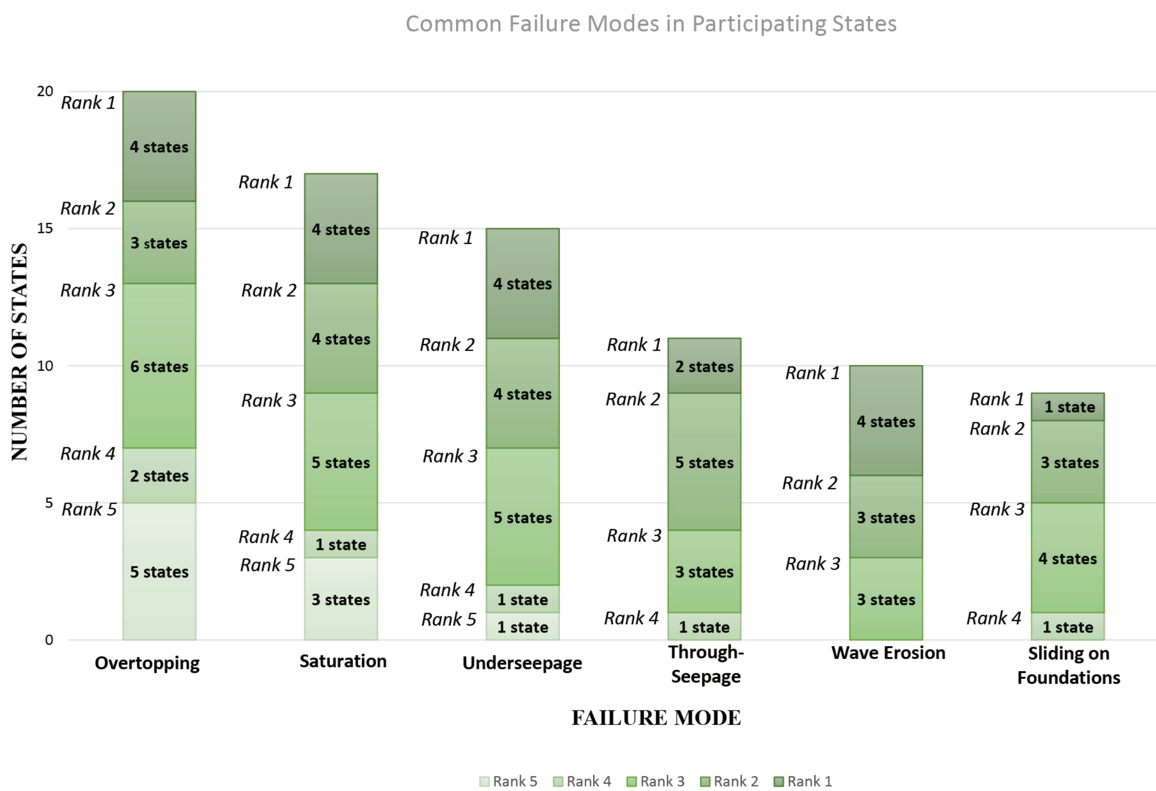

FIGURE 103 Common failure modes in participating states.

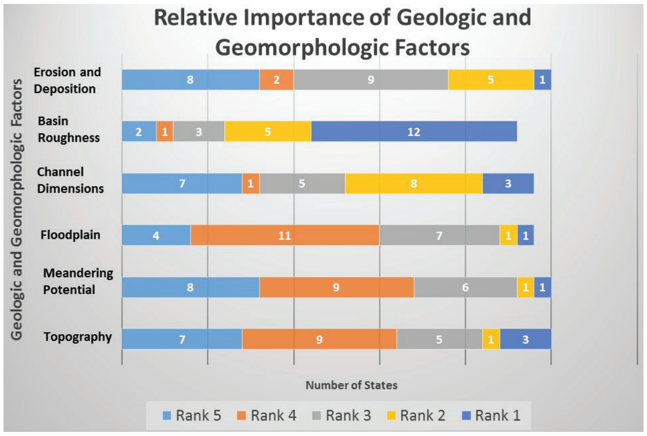

FIGURE 104 Relative importance of geologic and geomorphologic factors among states.

Additional notes were included related to what factors should be largely considered in design. River meandering and sedimentation/deposition processes directly affect the design for flooding, particularly armoring, maintenance and general dimensions of the embankment. The general topography and floodplain are considered critical factors in selecting the site location.

HYDRAULIC AND HYDROLOGIC CONSIDERATIONS

Q6. Please indicate the recurrence interval of the design flood/hurricane adopted for highway embankments.

Based on the survey results, the recurrence interval as well as the availability of guidelines for the selection of recurrence interval for a roadway embankment vary from state to state. The most common recurrence intervals vary between 25 and 100. A number of factors were used in different states to select a recurrence interval such as the road classification, the average annual daily traffic, urbanization, and the size of the watershed. Such factors are shown in Table 19.

It was also noted that some states do not have regulations specific to roadway embankments. In such cases, available guidelines for other drainage structures are used.

As for hurricane requirements, three states specified the 500-year recurrence interval.

Q7. How do you predict the recurrence interval (RI) that would lead to overtopping for an existing embankment?

Hydraulic analysis is commonly carried out among DOT engineers to identify the overtopping height and the flow velocities. About 70% of the participating states use HEC-RAS and 50% of them also use HY-8.

TABLE 19
DESIGN RECURRENCE INTERVAL REQUIREMENTS IN DIFFERENT STATES

State	Guidance for Selection for Recurrence Interval
Arkansas	Design for a 50-year storm and check for 100- year storm.
Connecticut	Assumed same requirements as those for hydraulic structures: 50-year storm and 100 years for drainage area > 1 mi. Check is required with 100-year and 500-year intervals, respectively.
Florida	Requirements for bridges: up to 50 years depending on the AADT (average annual daily traffic).
Georgia	Use a 50-year storm for the base, 100-year storm for shoulder breaking point.
Idaho	No overtopping of roadways allowed at the 199-year storm.
Illinois	Overtopping criteria are available: low point of roadway across the floodplain = Q50 + 3 ft; freeboard = 3f.
Maryland	Recurrence design criteria is dependent on road classification.
Minnesota	Minimum overtopping criteria varies with AADT and the type of the infrastructure. 100- year flood or "flood of record" have been used.
North Dakota	Administrative code requirements: 25-year storm for state highways, 50-year storm for interstate.
Utah	Different requirements (10–50 years) depending on highway classification and AADT.
Wyoming	100-year storm if developed property is involved.

Q8. How do you estimate the peak discharge corresponding to a flood recurrence interval for gaged and ungaged sites?

A number of methods are used by state DOTs to compute peak discharges for gaged and ungaged sites. Generally, a combination of methods is used. In gaged sites, about 60% of the participating DOTs use direct available data, about 40% use regression equations, about 28% use HEC-HMS, and about 10% use FEMA studies. In ungaged sites, a number of methods are also used to determine the peak discharges. About 52% use regression equations, 35% use StreamStats, and 24% use HEC-HMS.

Q9. When overtopping takes place, how long does it typically last?

The answers to this question were limited, typically because records of such data are not maintained by DOTs. Roughly, overtopping events could last anywhere between less than an hour to several weeks. The longest overtopping duration—2 to 3 weeks—was presented by Minnesota. Other respondents specified overtopping durations up to 48 hours. One answer was obtained relevant to the duration in coastal environments, which is 5 hours.

GEOTECHNICAL CONSIDERATIONS

Q10. What is the soil type typically used to build embankments?

Based on the survey results, all types of materials are used to construct roadway embankments. The materials requirements are generally specified in relevant state manuals. Consideration is given to use the site materials for feasibility purposes.

The types of embankment materials used is summarized in Figure 105; 32% of the participating states use both coarse and fine fill material, 27% use only coarse, 27% use only fine, and 14% use coarse grained, fine grained, and zoned embankments.

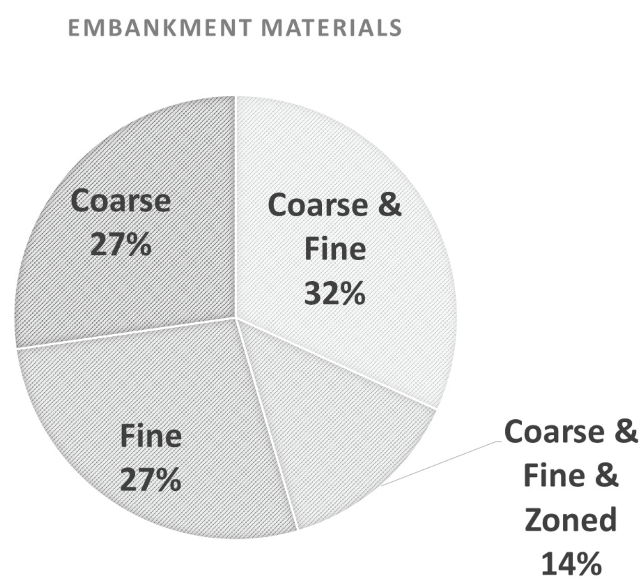

FIGURE 105 Embankment material.

Q11. Please indicate the in situ and/or lab tests used to design embankments subjected to flooding.

Limited soil testing data were received from the survey. This could be because the survey answers were generally completed by hydraulic engineers.

Based on the available data, soil testing was mostly limited to gradation, plasticity, and compaction. Two states specified the use of strength tests (shear and triaxial). One state specified the use of Pin Hole (erodible soils) and testing for strength of erosion control mats. One state further noted that it does not carry out any tests specifically for flood design.

DESIGN AND CONSTRUCTION CONSIDERATIONS

Q12. Please specify the chosen design life of embankments.

Different answers were given by the states. The answers ranged from as low as 5 years to as high as 50 years, 75 years, and 100 years because the embankment was expected to last as long as the structure.

Q13. Are there any special pavement or subbase requirements/techniques that are used to decrease the chance of pavement loss due to flooding?

Of the 29 answers given for this question, seven states said that they have some considerations for pavements prone to flooding. Those considerations are outlined herein:

- Using rockfill for the subbase to avoid scour/erosion (Arkansas Practice)
- Using black base, graded aggregate base, and underdrain (Florida)
- Allowing for a freeboard of 2 ft at design flood for a culvert (Idaho)
- Using recycled asphalt pavement, geogrid with vegetation, riprap, or cap clay on the slope, and paving the slope (Minnesota)
- Armoring the slope (Nevada)
- Keeping the subbase above design flood level (Tennessee).

PROTECTION TECHNIQUES

Q14. What current design, maintenance, and repair techniques do you use to minimize roadway embankment damage during flooding?

The protection systems commonly used for embankments are shown in Figure 106. Based on 30 answers, vegetation and riprap are the most commonly used protection means in 19 states (about 65%), followed by 12 states (40%) that use gabions, 11 (37%) that use precast concrete blocks, and five (17%) that use armoring.

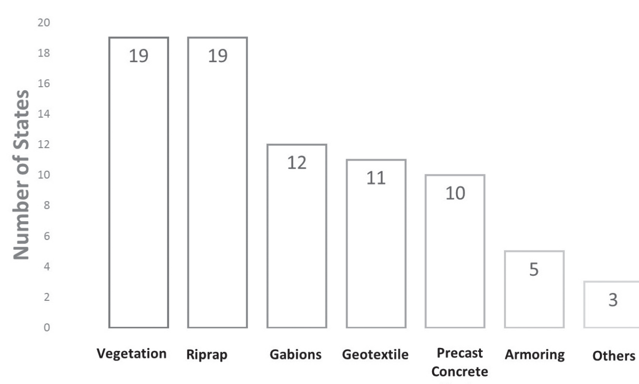

FIGURE 106 Protective measure installed by DOTs.

Obviously, the state practice is generally not limited to one protection measure but rather a number of measures for different applications (different applied velocities and different weather conditions when it comes to vegetation). Other practices were identified by three states: rock buttresses, concrete lining of slopes, and the use of fuse plugs. One state did not specify any protection measures. Instead, the importance of the application of an internal drainage system to reduce hydrostatic pressures was emphasized.

A comparison of the estimated costs and installation time between the protection systems is further presented in Figures 107 and 108, respectively. Based on the figures, the vegetation, the geotextile, and the rock buttresses are on the low cost end. The cost of riprap ranges from low to high depending on its size. Armoring and concrete lining fall in the intermediate cost range, while gabions and precast concrete blocks are on the high cost end. As for the installation time, the vegetation, riprap, geotextile, rock buttresses, and lining can be constructed in a short amount of time. Gabions and armoring require more installation time, and precast concrete blocks generally are the most time consuming.

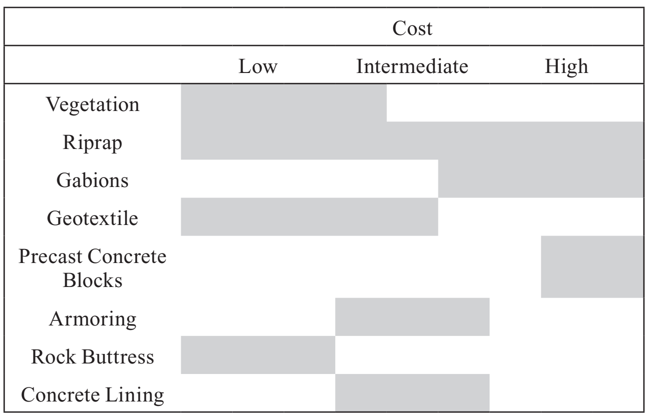

FIGURE 107 Comparison between the costs of protection systems.

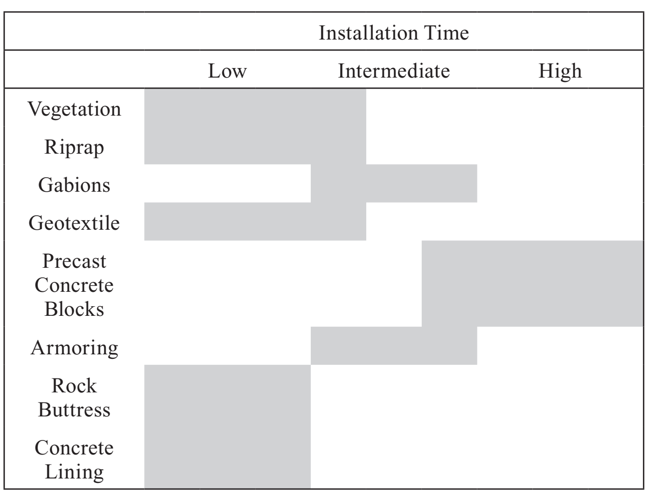

FIGURE 108 Comparison between the times of installation for the protection system.

It is important to note that geotextiles are generally used in combination with other systems such as under riprap or precast concrete blocks. And even though vegetation can be installed quickly, it requires more time to grow and develop its full resistance.

Q15. What type of vegetation, vegetation cover density, and height of the vegetation is used to protect highway embankments from damage during flooding?

Various types of vegetation are commonly used across the states. The selection of type depends on the weather, and specific seed mixes are generally developed for different weather conditions within each state. Aside from the different types of mixes available, different densities and heights are adopted by different states.

Q16. What sediment control techniques do you use upstream of highway embankments to capture sediments and minimize the potential for flooding?

Silt fencing is the most commonly used sediment control technique (17 answers), followed by bales (12), filter strips (nine), and, finally, retaining walls (one). It is worth noting that such techniques are not necessarily used to minimize embankment damage during flooding.

PROBABILITY AND RISK

Q17. What probability of failure does your agency consider acceptable for a highway embankment and for a pavement?

This section identified the current risk-related practice. Of the 23 states that answered this question, five indicated that they tie their design practice to risk. One state indicated that because roadway embankments are classified based on their average annual daily traffic, the higher this value the higher the protection required for the embankment. Another state indicated that informal risk evaluations are typically considered. This evaluation is tied to minimum safety factors outlined in the geotechnical manual.

DECISION-MAKING PROCESS AND FUNDING

Q19. Who is responsible for repairing highway embankment damage due to flooding in your state?

A number of agencies are consistently named as responsible for repairing damage, mainly the owner of the roadway and the DOT maintenance agency. One state clarified that a combination of forces are responsible depending on who owns the road.

Q20. Please specify the source of funding available in cases of emergency repair due to damage by flooding events.

The source of funding was identified by 12 states as ER and FEMA funds. One state commented that the source of funding depends on whether the area is declared a "disaster area" by the governor or president. Three states identified only ER as a funding agency. Six states specified state funds.

Q21. Sometimes, there may be a request by a related agency to design the embankment as a levee and this impacts the design or repair process. What are the factors that influence the design and/or repair decision process to minimize highway embankment damage during flooding?

Seven states clarified that FHWA discourages the use of roadways as levees. As a result, these states do not design embankments as levees. A number of considerations that would influence the design/repair decision process were mentioned, including considerations related to the funding agency, environmental considerations, and feasibility and time considerations, in addition to prioritization of other projects.

CURRENT AND FUTURE RESEARCH AREAS

This section aims to identify the documents currently used to minimize roadway embankment damage from flooding, as well as the areas that require future research.

Q22. Sometimes, there may be a request by a related agency to design the embankment as a levee and this impacts the design or repair process. What are the factors that influence the design and/or repair decision process to minimize highway embankment damage during flooding?

The survey respondents provided the following existing studies:

- Douglas and Krolak (2008), *Highways in the Coastal Environment, HEC-25,* 2nd ed.
- Chen and Anderson (1987), *Development of a Methodology for Estimating Embankment Damage Due to Overtopping,* FHWA/RD-86/126
- Paul and Clopper (1989), *Hydraulic Stability of Articulated Concrete Block Revetment Systems During Overtopping Flow,* FHWA-RD-89-199
- Clopper and Chen (1988), *Minimizing Embankment Damage During Overtopping Flow,* FHWA-RD-88-181

The survey also identified ongoing relevant research in Minnesota:

- MnDOT—Design Considerations for Embankment Protection during Road Overtopping Events, University of Minnesota—project in progress, end date March 31, 2017.
- MnDOT Flash Flood Vulnerability and Adaptation Assessment Pilot Project, Philip Schaffner. This project includes investigation of slope failure, maintenance-identified historical overtopping, and assessed impacts of climate change on overtopping frequency.
- NCHRP—NCHRP 24-36 Scour at the Base of Retaining Walls, David Reynaud—in progress.

Q23. Which areas of embankment damage due to flooding require further research?

The following issues were identified by the survey as requiring further research.

General design issues

- Recommendations for the design of approach embankments
- Recommendations for the design of culverts placed in embankments
- Guidance on designing inexpensive embankment protection at locations subject to overtopping
- Development of the tractive shear equation Meyer-Peter-Muller coefficients and the embankment length and time step of application
- Guidance for mass wasting or slip-circle failure of embankment
- The effect of tailwater level on embankment erosion
- The effect of fill height on embankment erosion
- The effectiveness of vegetation in resisting erosion
- Design of embankment gradation

Pavement-related research

- Preserving the pavements during flooding through the use of paved versus gravel shoulders

Risk-related guidelines

- Development of a risk-based methodology for prioritizing the implementation of climate change adaptation measures at state highway structure locations (especially at locations within the coastal zone)

Compliance with other agency requirements

- Means and methods of ensuring proposed embankment "resilience" in compliance with Amended EO 11988 and the Federal Flood Risk Management Standard

SUMMARY

This chapter presented a summary of the survey results obtained from DOT engineers. The replies received on each survey question were compiled and tabulated as applicable. These results, along with the information gathered from the case examples and the literature review, were used to outline design considerations, countermeasures, and mitigation and repair measures in the next two chapters.

CHAPTER NINE

DESIGN CONSIDERATIONS

INTRODUCTION

Roadway embankments do not play a flood control role; they may or may not be designed to resist flooding. If they are to be designed to resist flooding, helpful design steps specified in this chapter are consistent with best practice. This best practice is based on the national survey results, the available literature, and information from other fields such as levee and dam design. The purpose of the design is to minimize the probability of occurrence of the modes of failure identified in chapter two. These modes of failure are listed here:

1. Overtopping
2. Seepage through the roadway embankment
3. Seepage under the roadway embankment
4. Wave erosion
5. Softening by saturation
6. Lateral sliding
7. Culvert problems
8. Loss of pavement
9. Rapid drawdown.

The chart in Figure 109 summarizes the steps that may be followed to design against flooding for all the failure modes. Note that in several instances the current knowledge is limited and engineering judgment as well as prior experience will always be extremely valuable in the design process. This chapter proceeds by discussing each one of the design steps in more details.

CHOOSE THE DESIGN FLOOD

This is the step where the flood return period T_r, also called recurrence interval RI or even flood frequency F_f, is chosen. This is the most important step as it controls many of the subsequent calculations and decisions. As explained in the section "Modes of Failure" in chapter eight, the most effective approach to choose the design flood is the risk approach. Targeting a risk of 0.001 fatalities per year and $1,000 in the United States per year may be acceptable target risk values. The calculation of the design flood is based on the equation

$$R = PoE \times C \tag{9.1}$$

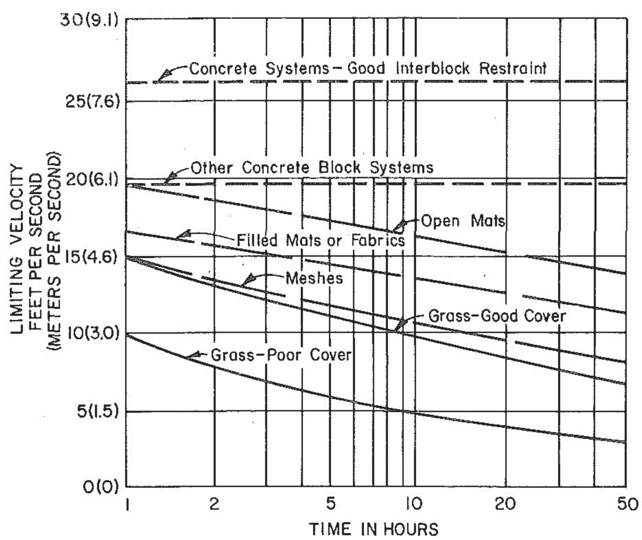

FIGURE 109 Recommended limiting values for erosion resistance of plain and reinforced grass (Hewlett et al. 1987).

Where R is the risk, PoE the probability of failure or exceedance, and C the value of the consequence. This equation can be used with the target values of R mentioned previously and the value of C obtained by evaluating the likely consequences of a failure of the embankment in terms of fatalities and cost. With R and C known, one can back-calculate the required PoE. Note that two values of the PoE will be obtained, one for fatalities and one for cost. The lower of the two values is used for further calculations.

Once the PoE is known, the following equation can be used to find the return period T_r of the design flood, which will satisfy the target risk.

$$PoE_Y = 1 - \left(1 - \frac{1}{T_r}\right)^Y \tag{9.2}$$

Where PoE_y is the target PoE coming from Equation 9.1 with y being the number of years for the design life of the embankment. Now that the return period of the flood is known, the design can proceed for each one of the failure modes identified in Figure 109.

OVERTOPPING

Calculate Discharge *Q*

This is done by using the regression equation available locally and for the return period identified in the previous section if there is no gage station close to the location being studied. If there is a USGS gage station at that location, the gage record is analyzed to identify the discharge, which corresponds to the return period identified in the previous section.

Calculate Water Depth

Once the discharge Q is known, the water depth in the river next to the embankment can be obtained by using a software program such as HER-RAS. The input includes the river cross section along with the roughness of the river bottom and the discharge. The water depth is output.

Compare Water Level and Embankment Height

If the water level is lower than the embankment height, no overtopping will occur for the design flood, and that part of the design is complete. If the water level is higher than the embankment height, overtopping will occur and the next step needs to be taken.

Calculate Maximum Velocity v_{max}

The maximum velocity is the velocity that occurs at the bottom of the downstream slope after overtopping takes place. This velocity can be conservatively estimated by

$$v_{\max} = \sqrt{2gh} \tag{9.3}$$

Where g is the acceleration from gravity and h is the vertical distance over which the water is falling on the downstream slope of the embankment. A closer and less conservative estimate of v_{max} is given by the equations in the chapter four section "Hydrological Methods and Considerations."

Compare Maximum Velocity and Critical Velocity

The value of v_{max} is then compared with the value of the critical velocity v_{crit} for the soil on the downstream slope. This critical velocity can be estimated by using Figure 66 in chapter five. If high-quality grass satisfying the criteria in "Overtopping of Embankments" in chapter five is in place, the critical velocity value may be increased. If v_{max} is lower than vcrit, then no erosion will occur during overtopping and the design is complete. If v_{max} is higher than v_{crit}, then a countermeasure is needed (see chapter 10).

SEEPAGE THROUGH THE EMBANKMENT

Draw Flow Net

A flow net is drawn through the embankment to represent the flow lines and equipotential of the flow.

Calculate the Exit Gradient i_e

This is done by calculating the hydraulic gradient in the smallest flow field on the exit face of the flow net (Briaud 2013).

Calculate the Critical Gradient i_c

This is done by using the equation

$$i_c = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} \tag{9.4}$$

Where γ_{sat} is the saturated unit weight of the soil in the embankment and γ_w is the unit weight of water. The critical hydraulic gradient value is often around 1.

Compare the Exit and Critical Gradients

If i_e is less than i_c divided by an appropriate factor of safety (2?), then the seepage through the embankment is unlikely to create backward erosion and loss of effective stress at the exit face. If i_e is approaching i_c or is larger than i_c, then a countermeasure is necessary.

Check for Internal Erosion and Piping

The design for this issue is not very precise and strong engineering judgment and experience is favored over a mathematical approach. One design aide for internal erosion is Figure 79 in chapter five.

SEEPAGE UNDER THE EMBANKMENT

The steps for this design issue are the same as the steps for seepage through the embankment listed above, except that in Equation 9.4, γ_{sat} is the saturated unit weight of the soil under the embankment rather than of the embankment.

WAVE EROSION

Waves can overtop the embankment at regular intervals during a storm without the embankment being subjected to constant sheet flow overtopping. In this case, the mean water level is lower than the embankment height but the waves can still overtop the crest. This case is similar to the sheet flow case except that the time over which the flow takes place is limited. The example in chapter five, "Example Process for Calculating Erosion Depth," shows how the calculations might proceed. In the end, the most effective approach is to select and design the proper countermeasure.

SOFTENING BY SATURATION

Check the Sensitivity of the Soil to an Increase in Water Content

This problem results from the loss of stiffness of the embankment soil as it becomes saturated by the water standing and flowing next to the embankment. This loss of stiffness can be checked by performing some Proctor compaction tests and obtaining the soil modulus as a function of the water content. If the soil shows little decrease in modulus with increasing water content, then this part of the design is complete. If, however, the soil exhibits a curve such as the one of Figure 8 in chapter two, then the soil is quite sensitive to an increase in water content.

Improve Soil

A soil less sensitive to water content increase must be chosen for the embankment.

Load Test Pavement After Flood

In case of uncertainty, the pavement can be load tested to check that the displacement under load is still acceptable before reopening the roadway embankment to traffic after the flood.

LATERAL SLIDING

Calculate the Water Push

The horizontal push P per unit length of embankment exerted by the water on the embankment is calculated from

$$P = \frac{1}{2}\gamma_w h^2 \tag{9.5}$$

Where γ_w is the unit weight of water and h is the water depth.

Calculate the Embankment Resistance

The horizontal resistance of the embankment at the level of the bottom of the water is a result of the friction that exists between the embankment and the soil underneath the embankment. This friction force is calculated by first calculating the shear strength of the interface between the embankment and the soil underneath it. This shear strength is equal to the mean vertical effective stress at the interface between the embankment and the soil below times the friction coefficient tanφ. That shear strength is then multiplied by the width of the embankment to obtain the maximum resistance R per unit length of embankment.

Calculate the Factor of Safety Against Sliding

The factor of safety against sliding is given by the ratio

$$F = \frac{R}{P} \tag{9.6}$$

If this factor of safety is satisfactory, sliding is unlikely. If this factor of safety is not large enough, countermeasures need to be implemented.

CULVERTS

Check Hydraulic Capacity

This is done by using the design discharge to calculate how much water will be directed through the culvert during the flood. If the culvert is not big enough, a second culvert may have to be installed or a bigger culvert is needed.

Erosion Around the Culvert

The boundary between the culvert and the surrounding soil is sensitive to erosion. It is important to ensure that good compaction is achieved at the boundary between the culvert and the surrounding soil.

PAVEMENTS

The best way to prevent partial or total pavement loss is to design an appropriate countermeasure (see chapter 10). In general, no unified guidelines have been adopted by all state DOTs to prevent pavement damage. Yet a number of practices gathered from the survey results are included herein:

- Using rockfill for the subbase to avoid scour/erosion (Arkansas Practice)
- Using black base, graded aggregate base, and underdrain (Florida)
- Allowing for a freeboard of 2 ft at design flood for a culvert (Idaho)
- Using recycled asphalt pavement, geogrid with vegetation, riprap, or cap clay on the slope, and paving the slope (Minnesota)

- Armoring the slope (Nevada)
- Keeping the subbase above design flood level (Tennessee).

Rapid Drawdown

This condition is related to the case where the flood recedes relatively quickly but the low permeability of the embankment soil causes the water stress in the embankment to be retained. In this situation, the soil strength is low and the water pressure is no longer there to help support the upstream slope. This is the rapid drawdown case in slope stability analysis.

Calculate Water Stress in the Upstream Slope

This is done by using the results from the "Seepage Through the Embankment" section in this chapter. The flow net is used to calculate the water stress in the upstream embankment slope.

Perform Slope Stability Analysis

The water stress distribution is part of the input in a slope stability analysis. The output of this analysis is a factor of safety *FS* against the sliding of the slope. If *FS* is satisfactory, rapid drawdown is not a problem. If *FS* is too low, a countermeasure is necessary.

SUMMARY

This chapter outlines useful design considerations based on the information gathered through the case examples, survey, and literature review. First the design flood is selected, then the approaches that could be adopted in the mitigation of each of the failure modes identified in chapter two is presented. The next chapter presents the countermeasures and maintenance solutions that would further minimize the roadway embankment damage from flooding.

CHAPTER TEN

COUNTERMEASURES, MAINTENANCE, AND REPAIR

INTRODUCTION

This chapter presents a compilation of the countermeasures, maintenance, and repair solutions adopted in current practice to minimize roadway embankment damage from flooding. All the failure modes specified in chapter two are addressed herein with the corresponding countermeasures based on the DOT engineers' feedback through the case examples, survey responses, and interviews. In the case of constructing a new roadway embankment, a key design element is selecting an adequate roadway location. This would be achieved through understanding the site characteristics and constraints (e.g., geology, geotechnical characteristics, stream hydraulic characteristics and meandering potential, upstream and downstream conditions, construction activities, and stormwater management facilities). In case of an old problematic site, it is essential to understand the key issue leading to recurring or aggravated damage. Possible issues include an increase in the severity of flooding events and stream instability issues caused by changes in upstream or downstream conditions (including man-made activities). Through highlighting the main issues, an adequate design approach would be adopted. Such approaches would include one or a combination of relocating the embankment, stabilizing the stream, or designing for failure modes as discussed herein.

The following references can be visited for relevant information:

- Clopper and Chen, (1988), *Minimizing Embankment Damage During Overtopping Flow,* FHWA-RD-88-181
- Richardson, Simons, and Lagasse (2001), *River Engineering for Highway Encroachments, Highway in the River Environments,* HDS-6, FHWA NHI 01-004
- Brown and Clyde (1989), *Design of Riprap Revetment, HEC-11,* FHWA-IP-89-016
- Kilgore and Cotton (2005), *Design of Roadside Channels with Flexible Linings, HEC-15,* 3rd ed., FHWA-NHI-05-114
- Lagasse, Zevenbergen, Spitz, and Arneson (2012), *Stream Stability Highway Structures, HEC-20,* 4th ed., FHWA-HIF-12-004
- Lagasse et al. (2009a), *Bridge Scour and Stream Instability Countermeasures, HEC-23,* Vol. 1
- Lagasse et al. (2009b), *Bridge Scour and Stream Instability Countermeasures, HEC-23,* Vol. 2
- Douglas and Krolak (2008), *Highways in the Coastal Environment, HEC-25,* Vol. 1
- Douglas, Webb, and Kilgore (2014), *Highways in Coastal Environment: Assessing Extreme Events, HEC-25,* Vol. 2
- Clopper (1989), *Hydraulic Stability of Articulated Concrete Block Revetment Systems During Overtopping Flow,* FHWA-RD-89-199
- Lagasse, Clopper, Zevenbergen, and Girard (2007), *Countermeasures to Protect Bridge Piers from Scour,* NCHRP Report 593
- Lagasse, Clopper, Zvenbergen, and Ruff (2006), *NCHRP Report 568: Riprap Design Criteria, Recommended Specifications, and Quality Control*
- Fay, Akin, and Shi (2012), *Cost-Effective and Sustainable Road Slope Stabilization and Erosion Control.*

OVERTOPPING

When overtopping is anticipated, mitigation measures or countermeasures, or a combination of both, could be adopted. Mitigation measures include adding culverts to prevent overtopping, raising the level of the embankment crest, and adopting fuse plugs. Countermeasures include protecting the downstream slope to minimize the potential of back erosion and, ultimately, the failure of the embankment using vegetation, riprap and geotextile, precast articulated concrete block matrices, gabions, and concrete lining of the slopes.

This section presents commonly used protection measures against overtopping. Clopper and Chen (1988) give further recommendations about the design and construction aspects of each system. Important considerations include selecting the extent of protection, ensuring that the slopes can carry the weight of the protection system, and preventing the erosion of soil particles through the protection system.

Vegetation

Based on the survey results, various seed mixes are used in roadway embankments based on weather and moisture conditions. The seed mixes are developed by relevant states, and although they are not designed for flooding resistance, they can help significantly in slowing down erosion as explained in "Overtopping of Embankments" in chapter five. To play an effective role in resisting erosion, however, the resulting vegetation has to be constantly maintained to satisfy the minimum requirements listed under "Overtopping of Embankments." Table 20 presents maximum permissible velocities and shear stresses for vegetative linings. Figure 109 presents a comparison between recommended limiting values for erosion resistance of plain and reinforced grass.

TABLE 20
MAXIMUM PERMISSIBLE VELOCITIES AND SHEAR STRESSES FOR VEGETATIVE LININGS

Cover (1)	Slope range (%) (2)	Permissible Velocity, fps (m/s)	
		Erosion-resistant soils (3)	Easily eroded soils (4)
Bermunda grass	0–5	8 (2.4)	6 (1.8)
	5–10	7 (2.1)	5 (1.5)
	>10	6 (1.8)	4 (1.2)
Buffalo grass, Kentucky Bluegrass, Smooth	0–5	7 (2.1)	5 (1.5)
Brome, Blue Grama	5–10	6 (1.8)	4 (1.2)
	>10	5 (1.5)	3 (0.9)
Grass mixture	0–5[a]	5 (1.5)	4 (1.2)
	5–10[a]	4 (1.2)	3 (0.9)
Lespedeza Sericea, Weeping Love Grass, Ischaemum (yellow bluestem), Kudzu, Alfalfa, Crabgrass	0–5[b]	3.5 (1.1)	2.5 (0.8)
Annual–used on mild slopes or as temporary protection until permanent covers are established, common lespedeza sudan grass	0.5[c]	3.5 (1.1)	2.5 (0.8)

[a] Do not use on slopes steeper than 10%.
[b] Do not use on slopes steeper than 5%, except for side slopes in a combination channel.
[c] Use on slopes steeper than 5% is not recommended.

Additional references include *NCHRP Report 430* (Fay et al. 2012), which refers to abundant literature relevant to the topic.

Riprap and Geotextile

Riprap has been successfully used for erosion protection in both coastal and riverine environments. Guidance for sizing riprap is available in a number of references including Brown and Clyde (1989), Richardson et al. (2001), Kilgore and Cotton (2005), Douglas and Krolak (2008), and Lagasse et al. (2006a, 2009a and b, 2012), and a typical riprap installation is shown in Figure 110.

It is important to place a filter between the soil and the riprap layer once applied on slopes, to protect the soil particles from erosion. Without a filter, the soil under the riprap may continue to erode through the large voids in the riprap. In the end, the riprap may not move away, but may simply move significantly as the underlying soil erodes away. The filter may be a sand filter or a geosynthetic filter. Relevant design guidelines can be found in Heibaum (2004) for sand filters and in Koerner (2012) for geosynthetic filters.

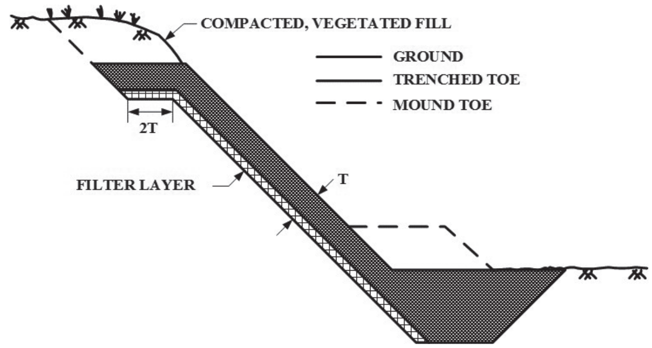

FIGURE 110 Typical riprap installation (after Brown and Clyde 1989).

Gabions (Wire-Enclosed Rock)

According to Powledge et al. (1989), geotextiles perform well with vegetation and when placed beneath any other form of surface protection. Gabions are wire baskets filled with rock; they have been used successfully for protection against erosion. Gabions are relatively expensive and are susceptible to wire failure from corrosion (Brown and Clyde 1989; Powledge et al. 1989).

Precast Articulated Concrete Block Blankets

Precast articulated concrete mats consist of precast blocks tied together to form a mat. Two types are commonly used today: the open block and the closed block. The open blocks have open cells that allow for grass growth, which further increases erosion resistance. These systems could withstand high-velocity flows up to 26 ft/s (7.9 m/s) on a cohesive subsoil without failure (Powledge et al. 1989). A typical section is shown in Figure 111.

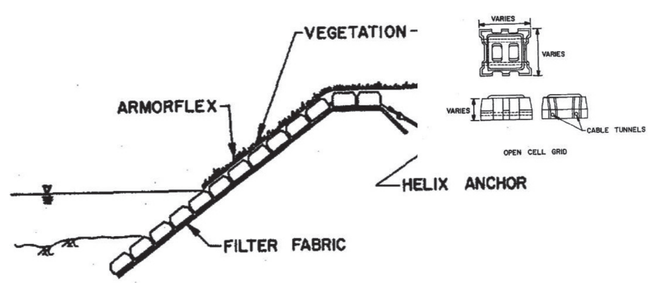

FIGURE 111 Section of a typical precast concrete blocks system (after Brown and Clyde 1989).

Concrete Lining

For concrete lining of the slopes, it is important to consider the possibility of a "blow-out" failure. Such failure is illustrated in Figure 112. This is the case of a possible blow-out failure in RM 335 in Real County, Texas. The blow-out could likely be the result of the hydrostatic pressure in the embankment.

FIGURE 112 Failure of concrete lining (RM-335, Real County, Courtesy of TxDOT).

THROUGH-SEEPAGE

To alleviate the hydraulic pressures developing within the embankment and possibly leading to erosion on the downstream slope, a pervious toe can be integrated into the system, as shown in Figure 83. If both through-seepage and underseepage are anticipated, a combined solution of a pervious toe and a partially penetrating toe trench can be adopted (Figure 84). Other horizontal and inclined drainage options are presented in Figure 85.

UNDERSEEPAGE-SEEPAGE

A number of measures are available to improve the drainage conditions within the embankment. Such measures include cutoff walls placed beneath the embankment, riverside blankets, downstream seepage berms (Figure 86), pervious toe trenches (Figure 87), and pressure relief wells.

WAVE-EROSION

The design procedures to protect embankments against wave action generally involve installing adequate revetments. The introduction to chapter three provides an example of the use of a combined solution in coastal environment that includes the protection of the seaward slope using sheet piles, soldier piles, articulate concrete blocks, and miscellaneous asphalt and performance turf. The installation takes place on the eroding slope as a result of wave action. Relevant guidelines include Douglas and Krolak (2008), USACE (2012), and Jones et al. (2005).

As shown in the chapter three section "Damage Due to Overtopping and Wave Action of Riverine Highways, Minnesota," paving the slopes is an option in certain cases. It is important to take measures to avoid rapid deterioration of the pavement and to extend the pavement sufficiently down the slope.

SOFTENING BY SATURATION

To overcome softening resulting from saturation, a soil that has a modulus insensitive to an increase in water content can be used.

LATERAL SLIDING

Lateral sliding can be minimized by using a keyway between the embankment and the natural soils. Using a flatter slope also improves the situation.

CULVERTS

HDS-5 discusses essential aspects of culvert design and provides additional references in this regard. Important considerations are relevant to erosion at inlet and outlet, aggradation, degradation and debris control, site-specific concerns, and structural considerations.

Erosion control measures at the inlets include slope paving, channel paving, headwalls, wing walls, and cutoff walls. Protection against scour at the outlets can vary from installation of end walls to simple riprap placement to installation of energy dissipation devices. Relevant information can be found in *HEC-14*.

Aggradation- and degradation-related problems are included in references such as *Highways in the River Environment* (1975). As for debris control measures that can be adopted for the culvert to function as designed, relevant discussion can be found in *HEC-9*. HDS-5 further discusses site-related modifications including the location and orientation of barrels, the case of multiple barrels, and the selection of culvert shape and materials.

Culverts could also face structural problems. For this purpose, proper selection of the bedding and fill are essential to resist the applied forces. Pipe bedding is a very important consideration and can vary from state to state. Culvert buckling under normally applied traffic loads could occur in dry conditions, while during flooding such problems as barrel floatation could be faced. A number of practical measures are listed in HDS-5. If such measures fail, anchoring the culvert might be a potential solution. In addition, the climate conditions can be considered in the design process. For instance, granular fill is often used to reduce frost heave and potentially improve compaction.

PAVEMENTS

A number of practices were extracted from the survey results that would decrease the impact of flooding on pavements:

- Using rockfill for the subbase to avoid scour/erosion (Arkansas Practice)
- Using black base, graded aggregate base, and underdrain (Florida)
- Allowing for a freeboard of 2 ft at design flood for a culvert (Idaho)
- Using recycled asphalt pavement, geogrid with vegetation, riprap, or cap clay on the slope, and paving the slope (Minnesota)
- Armoring the slope (Nevada)
- Keeping the subbase above design flood level (Tennessee).

STREAM STABILIZATION

If stream stabilization is adopted to minimize roadway embankment damage, a number of stabilization methods could be used as shown in the chapter three sections “MD-24 Deer Creek Stream Stabilization, Maryland” and “Kimsey Run Project, West Virginia.” Such methods include armoring the bank with riprap or implicated stone wall, or using tree trunks to strengthen the bank. Using in-stream structures such as cross vanes and rock vanes in the channel would contribute to decreasing the velocity of the water in the stream and divert it away from the embankment.

SUMMARY

This chapter presented the different countermeasures and mitigation solutions for each of the failure modes identified in chapter two, which are generally adopted in current practice. This information is based on the successful and failed case examples (chapter two) and the survey (chapter eight). The selection of an adequate measure is site-dependent. In other words, what works at one site does not necessarily work at another site.

CHAPTER ELEVEN

STATE OF RESEARCH AND CONCLUSIONS

INTRODUCTION

Information relevant to minimizing roadway embankment damage from flooding is limited. Based on the data gathered from the survey, the number of relevant ongoing studies is limited as well. Throughout the synthesis, research gaps have been identified that are related to a number of aspects. The ongoing studies and the future research needs are highlighted herein and the synthesis is concluded.

ONGOING STUDIES

Based on the survey results, three ongoing studies are relevant to the synthesis topic. Two of these projects are carried out for Minnesota Department of Transportation (MnDOT) and are related to design considerations for protection against overtopping and assessment of flashflood and related damage, maintenance, and the impact of climate change. The third study is carried out for NCHRP and addresses the case of scour at the base of retaining walls. The following are the references for these studies:

- MnDOT—Design Considerations for Embankment Protection During Road Overtopping Events, University of Minnesota—project in progress; end date March 31, 2017.
- MnDOT—Flash Flood Vulnerability and Adaptation Assessment Pilot Project, Philip Schaffner. This project includes investigation of slope failure, maintenance-identified historical overtopping, and assessed impacts of climate change on overtopping.
- NCHRP—NCHRP Project 24-36: Scour at the Base of Retaining Walls, David Reynaud—in progress.

FUTURE RESEARCH NEEDS

The future research needs are compiled in this section based on the feedback received from DOT engineers through the survey responses and interviews. The research needs could be grouped under the following topics: historical learning, geomorphic and geologic factors, protection systems, risk factors, guidelines for culverts and pavement considerations, factors related to management, and modeling packages.

Historical learning is essential to advancing our knowledge relevant to minimizing roadway embankment damage resulting from flooding. However, adequate documenting the flooding event occurrences and their effects is not commonly done as a part of DOT practice. When embankments and pavements are damaged or fail as a result of flooding, restoring service to the facility and studying the effects of the measures adopted on the budget become the main concern. As a result, documentation of the case is not regarded as a priority. Noting that the embankment and pavement failures from inundation often occur months after the event itself, by the time the damage is inflicted, the event details are already forgotten. This calls for the development of documentation guidelines and the preservation of that documentation.

Based on information from different case examples, knowledge of the geomorphologic and geologic setting is crucial to carry out adequate design for such projects. Understanding of the global situation based on aerial photographs and studying the different factors involved before making decisions cannot be further stressed. As a result, it would be helpful to highlight the geological and geomorphological factors that are important, and the effect of different relevant conditions on the design. Also, it would be important to emphasize the benefit of involving geological and geotechnical considerations in early design stages. Also, changing the floodplain owing to meandering or to a sudden change in a river course is not uncommon. It would be helpful to inspect why rivers change their course and predict such change.

Although general knowledge is available on the offered protection products, there is a shortage in documented expertise on the applicability and the success or failure of these systems. Also, it would be useful to establish borderlines between different types of protection systems through identifying at what point a costlier system can be adopted. Additionally, specific studies on available protection systems such as grouted riprap versus ungrouted riprap would be helpful. In this regard, another point of research would be the impact of debris in the channel on the riprap sizing, especially when larger debris is being transported.

It is common to see risk as one of the topics included in state requirements and federal guidelines and standards. Yet there is still no clear guidance on what level of risk is acceptable, and how to incorporate this into the design. Hence, it is important to develop a risk-based methodology that could be adopted in the design.

Because culverts are an important infrastructure in roadway embankments, standard clear guidance on the design and installation of culverts is essential. Different DOTs have different requirements for culverts and the case examples revealed several culvert-related failures in different locations. It is important to identify best practices applied in DOTs. Relevant to pavement, little guidance is generally available. It would be helpful to identify the effect of inundation on the pavement, how long it takes to regain its strength after saturation, and general preparation and installation guidelines.

Aside from the technical aspect, management is a very important factor in a project success. Especially when federal funds are used, it is important to finish the construction in time; otherwise, funding may be lost. Delays throughout the project may result from regulations imposed by different agencies (such as federal regulations, environmental regulations, and permitting regulations) that the engineer gradually becomes aware of throughout the project's course. Because not all relevant regulations are identified at the beginning of the project, meeting the construction deadlines might be compromised and compliance with agency requirements become challenging. It is also essential to have clear guidance on the studies that would be required for each project at an early stage. This will aid in better management and result in optimized designs.

Lastly, in order for design information to be useful for engineers, it would be helpful to present it for them as part of their software packages. This would facilitate its incorporation into the design. An example is developing software to predict meandering potential and future location of the channel. Also, software for modeling embankment damage is long overdue. Modeling of damage is important for many reasons, such as to identify whether a system is feasible and to predict at what point the embankment would fail.

CONCLUSIONS

Minimizing roadway embankment damage from flooding requires knowledge of many disciplines. Understanding the project needs based on the site-specific conditions at an early stage of design, the limitations of the available methodologies, the agencies involved and the relevant constraints on the decision-making process, the risk that can be accepted, and the relevant regulations on local, state, and federal levels are crucial for preparing adequate repair measures within the imposed time, monetary, and quality frame of work.

It is essential to understand that sound engineering judgment is an indispensable component of every design decision. Whereas this synthesis highlights some common practices and important design aspects, it is not a standard or a guideline. Rather, this synthesis is a first step toward understanding the components of this topic, employing available knowledge to produce better designs, and identifying the research gaps.

In summary, roadway embankment damage from flooding is a shared concern among the states. Aside from the financial burden the states and federal government face to repair the damage, the preparation of an adequate design is a challenging task. This synthesis highlights major issues and design components in the absence of standard guidance specific to this topic. The information presented in the synthesis is based on a review of the related literature, a survey of current practice, and a series of telephone interviews. The probable failure mechanisms are identified and possible design approaches and repair countermeasures are highlighted.

The study presents a comparison between roadway embankments and levees to emphasize that embankments are not designed as flood control structures. The differences between riverine and coastal flood mechanisms are also stated. The common failure mechanisms in coastal and riverine environments are identified as overtopping, seepage (through-seepage and underseepage), piping, wave action, softening by saturation, and lateral sliding on the foundation soil. Pavement failures and culvert-related failures are also outlined. Examples of failures and repair techniques are illustrated through 14 case examples gathered from six states.

As reflected by the case examples, variable state practices, and available literature, minimizing damage to roadway embankments can be tackled by altering the embankment design and slope protection techniques or altering the stream course, or both. A number of systems and approaches can be considered for the same project. The success of an approach is site-dependent because, as shown in the case examples, an approach that serves its intended design purpose at one site does not necessarily work at another site.

Different failure modes were identified by the case examples and different solutions were demonstrated. The effectiveness of the adopted solutions are generally dependent on the site conditions. A protection technique that would prove successful in one site condition is not necessarily the adequate solution for other site conditions. To arrive at an adequate design, the following factors should be considered: hydrologic and hydraulic factors, geological and geotechnical factors, legal and funding aspects, and risk. General observations and guiding principles are listed.

Hydrologic studies are essential in estimating the magnitude of the expected floods and in selecting a design flood. This information is used in nearly every aspect of the design. Hydrographs give information about the variation of the flow versus time. The use of hydrographs instead of peak flows can lead to more advanced analyses. Hydraulic methods use the hydrologic data to give an estimate of the water surface elevation, the overtopping height, and the water velocity. Major geological considerations include the subsurface conditions, topography, floodplain and meandering potential, erosion and deposition, and basin characteristics and channel dimensions. Such considerations are essential in identifying the expected sources of damage at an early stage. Geotechnical calculations are crucial in designing against anticipated failure modes. Embankments are made of soil; thus, the identification of the characteristics of the embankment materials and their impact on the behavior of the embankment during flooding is very important. Some key aspects include erodibility of the embankment materials, material properties (strength and permeability), and culvert- and pavement-related considerations.

The decision-making process is not solely based on technical aspects; it is also influenced by legal, regulatory, and funding aspects. After a failure, decisions must be made related to the type of repairs and to whether the same design will be repeated or whether betterments will be sought (temporary versus permanent, changing stream course, raising the freeboard). These decisions are all bound by funding constraints, time constraints, constraints from interaction with other agencies (such as the U.S. Army Corps of Engineers, Federal Emergency Management Agency, or Environmental Protection Agency), and, in some cases, community constraints.

Available design steps to mitigate the impact of flooding on roadway embankments are outlined. The steps include the following:

1. Choosing the design flood
2. Overtopping
3. Seepage through the embankment
4. Seepage under the embankment
5. Wave erosion
6. Softening by saturation
7. Lateral sliding
8. Culverts
9. Pavements
10. Rapid drawdown.

The main countermeasures are identified and their use is associated with the most effective applications. They include the following:

1. Vegetation
2. Riprap and geotextiles
3. Gabions
4. Articulated concrete blocks
5. Paving.

It is essential to couple the design process and the use of countermeasures with engineering judgment, while keeping in mind all the issues outlined in this synthesis. It is also important to recognize that no design is foolproof and that the probability of failure is not zero. Consequently, it is also important to evaluate the probability of failure and the value of the consequence in terms of lives lost and economic loss. Ideally, it is the combination of the probability of failure and the value of the consequence or risk that can most effectively guide the decision. Finally, the ongoing and future research needs are outlined.

UNITS

Acceleration	9.81 m/s^2 = 386.22 in./s^2 = 32.185 ft/s^2, Paris: g = 9.80665 m/s^2, London: g = 3.2174 x 101 ft/s^2
Area	1 m^2 = 1.5500 x 10^3 in.^{2} = 1.0764 x 10^1 ft^2 = 1.196 yd^2 = 10^6 mm^2 = 10^4 cm^2 = 2.471 x 10^4 acres = 3.861 x 10^7 mi^2 = 1.0000 x 10^{-4} hectares
Bending stiffness	1 kN.m^2 = 10^3 N.m^2 = 10^6 kN.mm^2 = 2.4198 x 10^3 lb.ft^2 = 2.4198 kip.ft^2 = 3.4845 x 10^2 kip.in.^{2} = 3.4845 x 10^5 lb.in.^{2}
Coefficient of consolidation	1 m^2/s = 3.1557 x 10^7 m^2/yr = 10^4 cm^2/s = 6x 10^5 cm^2/min = 3.6 x 10^7 cm^2 /h = 8.64 x 10^8 cm^2/day= 2.628 x 10^{10} cm^2/month = 3.1536 x 10^{11} cm^2/year = 1.550 x 10^3 in.^{2}/s = 4.0734 x 10^9 in^2/month= 1.3392 x 10^8 in^2 /day = 4.8881 x 10^{10} in.^{2}/year = 9.3000 x 10^5 ft^2/day = 2.8288 x 10^7 ft^2/month = 3.3945 x 10^8 ft^2/year
Flow	1 m^3/s = 10^6 cm^3/s = 8.64 x 10^4 m^3 /day = 8.64 x 10^{10} cm^3/day = 3.5314 x 10^1 ft^3/s = 3.0511 x 10^6 ft^3/day
Force	10 kN = 2.2481 x 10^3 lb = 2.2481 kip = 1.1240 t (short ton = 2,000 lb) = 1.0197 x 10^3 kg = 1.0197 x 10^6 g = 1.0197 T (metric ton= 1000 kg) = 10^9 dynes = 3.5969 x 10^4 ounces = 1.022 tl (long ton = 2200 lb)
Force per unit length	1 kN/m = 6.8522 x 10^1 lb/ft = 6.8522 x 10^{-2} kip/ft = 3.4261 x 10^{-2} t/ft = 1.0197 x 10^2 kg/m = 1.0197 x 10^{-1} T/m
Length	1 m = 3.9370 x 10^1 in. = 3.2808 ft = 1.0936 yd = 10^{10} Angstrom = 10^6 microns = 10^3 mm = 10^2 cm = 10^{-3} km = 6.2137 x 10^{-4} mile = 5.3996 x 10^{-4} nautical mile
Moment or energy	1 kN.m = 7.3756 x $l0^2$ lb.ft = 7.3756 x 10^{-1} kip.ft = 3.6878 x 10^{-1} t.ft = 1.0197 x 10^3 g.cm = 1.0197 x 10^2 kg.m = 1.0197 x 10^{-1} T.m = 10^3 N.m = 10^3 Joule
Moment of inertia	1 m^4 = 2.4025 x 10^6 in^4 = 1.1586 x 10^2 ft^4 = 1.4304 yd^4 = 108 cm^4 = 10^{12} mm^4

Moment per unit length	1 kN.m/m = 2.2481 x 10^2 lb.ft/ft = 2.2481 x 10^{-1} kip.ft/ft= 1.1240 x 10^{-1} t.ft/ft = 1.0197 x 10^2 kg.m/m = 1.0197 x 10^{-1} T.m/m
Pressure	100 kPa = 10^2 kN/m^2 = 1.4504 x 10^1 lb/in.^{2} = 2.0885 x 10^3 lb/ft^2 = 1.4504 x 10^{-2} kip/in.^{2} = 2.0885 kip/ft^2 = 1.0443 t/ft^2 = 7.5006 x 10^1 cm of Hg (0 °C) = 1.0197 kg/cm^2 = 1.0197 x 10^1 T/m^2 = 9.8692 x 10^{-1} Atm = 3.3489 x 10^1 ft of H_2O (60 °F) = 1.0000 bar = 10^6 dynes/cm^2
Temperature	°C = 5/9 (°F - 32), °K =°C + 273.15
Time	1 yr = 12 mo. = 365 day = 8,760 h = 5.256 x 10^5 min = 3.1536 x 10^7 s
Unit weight, coefficient of subgrade reaction	10 kN/m^3 = 6.3659 x 10^1 lb/ft^3 = 3.6840 x 10^{-2} lb/in.^{3} = 1.0197 g/cm^3 = 1.0197 T/m^3 = 1.0197 x 10^3 kg/m^3
Velocity or permeability	1 m/s = 3.6 km/h = 2.2369 mile/h = 6 x 10^1 m/min = 10^2 cm/s = 3.15 x 10^7 m/yr = 1.9685 x 10^2 ft/min = 3.2808 ft/s = 1.0346 x 10^8ft/year = 2.8346 x 10^5 ft/day
Volume	1 m^3 = 6.1024 x 10^4 in.^{3} = 3.5315 x 10^1 ft^3 = 1.3080 yd^3 = 10^9 mm^3 = 10^6 cm^3 = 10^3 dm^3 = 33814.02 ounces= 2113.38 pints (US) = 10^3 liter = 2.1997 x 10^2 gallon (UK) = 2.6417 x 10^2 gallon (US)

GLOSSARY

Aggradation—General and progressive buildup of the longitudinal profile of a channel bed from sediment deposition.

Articulated concrete block—Concrete slabs that can move without separating mattress: as scour occurs; usually hinged together with corrosion-resistant cable fasteners; primarily placed for lower bank protection.

Backfill—Material used to refill a ditch or other excavation, or the process of doing so.

Bedrock—Solid rock exposed at the surface of the earth or overlain by soils and unconsolidated material.

Discharge—Volume of water passing through a channel during a given time.

Downstream slope—Embankment slope that would be reached by water only if the embankment were overtopped.

Embankment—A raised earth structure on which the roadway pavement structure is placed.

Erosion—Displacement of soil particles from water or wind action.

Floodplain—A nearly flat, alluvial lowland bordering a stream, that is subject to frequent inundation by floods.

Freeboard—Vertical distance between the level of the water surface at design flow and a specified point (e.g., a bridge beam, levee top, location on a highway grade).

Gabion—Basket or compartmented rectangular container made of wire mesh. When filled with cobbles or other rock of suitable size, the gabion becomes a flexible and permeable unit with which flow- and erosion-control structures can be built.

Geomorphology—That science that deals with the form of the earth, the morphology: general configuration of its surface and the changes that take place as a result of erosion and deposition.

Headwater level—Level of water at the upstream of the embankment.

Hydrograph—Graph of stage or discharge against time.

Hydrology—Science concerned with the occurrence, distribution, and circulation of water on the earth.

Landside slope—Embankment slope that faces land in a coastal environment.

Levee—Embankment that plays a flood control role and prevents overflow from the wet side to the dry side.

Meandering stream—Stream having a sinuosity greater than some arbitrary value. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops. The channel generally exhibits a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.

Rapid drawdown—Lowering the water against a bank more quickly than the bank can drain without becoming unstable.

Riprap—Layer or facing of rock or broken concrete that is dumped or placed to protect a structure or embankment from erosion; also the rock or broken concrete suitable for such use. Riprap has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs.

Roadway—Portion of a highway, including shoulders, for vehicular use. (A divided highway has two or more roadways.)

Runoff—That part of precipitation that appears in surface streams of either perennial or intermittent form.

Seaside slope—Coastal embankment slope that faces the sea.

Seepage—Slow movement of water through small cracks and pores of the bank material.

Tailwater level—Water level at the downstream of the embankment.

Upstream—Side of the embankment from which water flows.

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APPENDIX A

Survey Questionnaire

Dear Survey Respondents:

The Transportation Research Board (TRB) is preparing a synthesis on minimizing roadway embankment damage from flooding. This is being done for NCHRP, under the sponsorship of the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration. An 80% response rate is required.

This questionnaire is part of NCHRP Synthesis Topic 46-16 to gather information on minimizing roadway embankment damage from flooding. We are interested in the experience and opinions of your agencies regardless of their direct or indirect involvement in embankment projects.

In order to maximize the benefit from the questionnaire, **please answer the questions and provide relevant references.** We accept all forms of references including hyperlinks, photos, videos, soft copies, and hard copies. In addition to the spaces provided herein for hyperlinks, other supporting materials (soft copies or hard copies) can be sent directly to Professor Jean-Louis Briaud by e-mail or at the postal address shown below:

Jean-Louis BRIAUD, PhD
Professor and Holder of the Buchanan Chair
Zachry Dept. of Civil Engineering
Texas A&M University
College Station, TX 77843-3136, USA Tel: 979-845-3795
Cell: 979-777-1692
E-mail: briaud@tamu.edu

This questionnaire is being sent to Professor Jean-Louis Briaud. Your cooperation in completing the questionnaire will ensure the success of this effort. **If you are not the appropriate person at your agency to complete this questionnaire, please forward it to the correct person.**

Please compete and submit this survey and the relevant references by the 15th of February, 2015. The questionnaire has 22 questions and we estimate that it should take approximately 30 minutes to complete. If you have any questions, please contact Professor Jean-Louis Briaud.

QUESTIONNAIRE INSTRUCTIONS

1. **To view and print the entire questionnaire prior to starting the survey:** //surveygizmolibrary.s3.amazonaws.com/library/64484/NCHRP_Synthesis_4616.pdf

2. **To view and print the entire questionnaire:** Click on the following link and print using “Ctrl” + “P”

3. **To save your partial answers and complete the questionnaire later:** Click on the “Save and Continue Later” link in the upper-right-hand corner of your screen. A link to the incomplete questionnaire will be e-mailed to you from SurveyGizmo. To return to the questionnaire later, open the e-mail from SurveyGizmo and click on the link. We suggest using the “Save and Continue Later” feature if there will be more than 15 minutes of inactivity while the survey is open, as some firewalls may terminate due to inactivity.

4. **To pass a partially completed questionnaire to a colleague:** Click on the on the “Save and Continue Later” link in the upper-right-hand corner of your screen. A link to the incomplete questionnaire will be emailed to you from SurveyGizmo. Open the e-mail from SurveyGizmo and forward it to a colleague.

5. **To view and print your answers before submitting the survey:** Click forward to the page following question 22. Print using “Ctrl” + “P”

6. **To submit the survey:** Click on “Submit” on the last page.

QUESTIONNAIRE DEFINITIONS

For the purposes of this survey, the following definition is used:

- **Flooding:** Flooding occurs when the water flows over the top of the embankment and generates potential erosion.

Thank you very much for your time and expertise.

Please enter the date (MM/DD/YYYY).

Please enter your contact information.

First Name*: Last Name*: Title: ______________________________

Agency/Organization*: ______________________________

Street Address*: ______________________________

Suite: City: State*: ______________________________

Zip Code*: Country: ______________________________

E-mail Address*: ______________________________

Phone Number*: ______________________________

Fax Number: ______________________________

Mobile Phone: ______________________________

URL: ______________________________

SECTION I: CASE EXAMPLES

1) **Please provide up to three successful case examples (and photos/videos if available) of a highway embankment subjected to flooding and why you think it was successful. Successful means no damage or limited damage. Please provide relevant contact information that will assist in a follow-up interview and a brief description of the project under the “Comments” section.**

Case example -1- Project Name: ______________________________

Comments:

Environment:

( ) Riverine

( ) Coastal

Pavement Details:

( ) Paved Roadways

( ) Unpaved Roadways

Available Information:

[ ] Hydrological

[ ] Geotechnical

[ ] Geological

[ ] Plans & Sections

[ ] Relevant Literature

[ ] Other(s): ____________________

Case example -2- Project Name: ____________________

Comments:

Environment:

( ) Riverine

( ) Coastal

Pavement Details:

( ) Paved Roadways

( ) Unpaved Roadways

Available Information:

[ ] Hydrological

[ ] Geotechnical

[ ] Geological

[ ] Plans & Sections

[ ] Relevant Literature

[ ] Other(s): ____________________

Case example -3- Project Name: ____________________

Comments:

Environment:

( ) Riverine

( ) Coastal

Pavement Details:

( ) Paved Roadways

( ) Unpaved Roadways

Available Information:

[ ] Hydrological

[ ] Geotechnical

[ ] Geological

[ ] Plans & Sections

[ ] Relevant Literature

[ ] Other(s): ____________________

2) **Please provide up to three failure case examples (and photos/videos if available) of a highway embankment subjected to flooding and why you think it failed. Failure means significant damage or complete destruction. Please provide relevant contact information that will assist in a follow-up interview and a brief description of the project under the "Comments" section.**

Case example -1- Project Name: ____________________________

Comments:

Environment:

( ) Riverine

( ) Coastal

Pavement Details:

( ) Paved Roadways

( ) Unpaved Roadways

Extent of Damage:

[ ] Full Breach

[ ] Partial Erosion

[ ] Pavement Damage

[ ] Other(s): ____________________________

Cause of Damage:

[ ] Overtopping

[ ] Internal Erosion

[ ] Wave Erosion

[ ] Softening due to Saturation

[ ] Failure due to Sliding on Foundations

[ ] Underseepage

[ ] Other(s): ____________________________

Case example -2- Project Name: ____________________________

Comments:

Environment:

( ) Riverine

( ) Coastal

Pavement Details:

( ) Paved Roadways

( ) Unpaved Roadways

Extent of Damage:

[ ] Full Breach

[ ] Partial Erosion

[ ] Pavement Damage

[ ] Other(s): ____________________

Cause of Damage

[ ] Overtopping

[ ] Internal Erosion

[ ] Wave Erosion

[ ] Softening due to Saturation

[ ] Failure due to Sliding on Foundations

[ ] Underseepage

[ ] Other(s): ____________________

Case example -3- Project Name: ____________________

Comments:

Environment;

( ) Riverine

( ) Coastal

Pavement Details:

( ) Paved Roadways

( ) Unpaved Roadways

Extent of Damage:

[ ] Full Breach

[ ] Partial Damage

[ ] Pavement Damage

[ ] Other(s) ____________________

Cause of Damage:

[ ] Overtopping

[ ] Internal Erosion

[ ] Wave Erosion

[ ] Softening due to Saturation

[ ] Failure due to Sliding on Foundations

[ ] Underseepage

[ ] Other(s) ____________________

SECTION II: REQUEST FOR DOCUMENTS

3) **Do you have hyperlinks, hard copies, and/or soft copies of the following documents? Please check the documents provided and fill in the reference(s). You can provide us with the references through hyperlinks pasted under the "Comments" section, through the "Upload" provided at the end of this question, and through e-mail/mail to Professor Jean-Louis Briaud provided at the beginning of the survey.**

Design documents that your agency currently uses for highway embankments subjected to flooding.

( ) Yes [Please provide reference(s)]:

____

( ) No

List of design criteria to be addressed in the design of highway embankments subjected to flooding.

( ) Yes [Please provide reference(s)]:

____

( ) No

Other design documents that you consider helpful in the design of highway embankments subjected to flooding.

( ) Yes [Please provide reference(s)]:

____

( ) No

Specifications used by your agency for embankment construction in flood-prone areas.

( ) Yes [Please provide reference(s)]:

____

( ) No

Sources of hydraulic and hydrologic information (e.g., discharge, precipitation, wave height) used by your agency in the design of highway embankments subjected to flooding.

( ) Yes [Please provide reference(s)]:

____

( ) No

Sources of geologic and geotechnical information used by your agency in the design of highway embankments subjected to flooding.

( ) Yes [Please provide reference(s)]:

____

( ) No

Documents used by your agency to assess embankment conditions for flood resistance.

( ) Yes [Please provide reference(s)]:

____

( ) No

Documents used by your agency to obtain funding for redesign, maintenance, and repair of an embankment damaged by flooding.

( ) Yes [Please provide reference(s)]:

____

( ) No

Guidance documents on legal and regulatory factors that play a role in the design, maintenance, and repair of highway embankments subjected to flooding.

( ) Yes [Please provide reference(s)]:

____

( ) No

Please upload the documents you have:

____1

SECTION III: MODES OF FAILURE

4) Please identify the failure modes of highway embankments due to flooding that your agency has experienced. Please rank them from the least common (1) to the most common (5). Also, please add your comments (if any) at the bottom of this question.

	Experienced		Rank				
	Yes	No	1 (least common)	2	3	4	5 (most common)
Overtopping Flow	( )	( )	( )	( )	( )	( )	( )
Wave Erosion	( )	( )	( )	( )	( )	( )	( )
Internal Erosion	( )	( )	( )	( )	( )	( )	( )
Softening by Saturation	( )	( )	( )	( )	( )	( )	( )
Underseepage	( )	( )	( )	( )	( )	( )	( )
Failure due to Sliding on Foundations	( )	( )	( )	( )	( )	( )	( )
Other(s), specify under the "Comments" section	( )	( )	( )	( )	( )	( )	( )

Comments: ____

PART IV: GEOLOGICAL CONSIDERATIONS

5) What are the geologic and geomorphologic factors that should be addressed in the design of highway embankments subjected to flooding and why? From the following list, please check all that apply and rank from the most important factor (1) to the least important factor in minimizing embankment damage from flooding. Space is provided for your comments (if any).

	Rank				
	1 (Least Important)	2	3	4	5 (Most Important)
Topography	( )	( )	( )	( )	( )
Meandering Potential	( )	( )	( )	( )	( )
Floodplain	( )	( )	( )	( )	( )
Channel Dimensions	( )	( )	( )	( )	( )
Basin Roughness	( )	( )	( )	( )	( )
Erosion and Deposition	( )	( )	( )	( )	( )
Other(s), specify under the "Comments" section	( )	( )	( )	( )	( )

Comments: ____

PART V: HYDRAULIC AND HYDROLOGIC CONSIDERATIONS

6) Please indicate the recurrence interval of the design flood/hurricane adopted for highway embankments. Please provide relevant references (and any comments) under the "Comments" section.

	Type of Storm	
	Flood	Hurricane
2 years	( )	( )
10 years	( )	( )
25 years	( )	( )
50 years	( )	( )
100 years	( )	( )
500 years	( )	( )
Other(s), specify under the "Comments" section	( )	( )

Comments: ____

7) How do you predict the flood recurrence interval (RI) that would lead to overtopping for an existing embankment? Please specify the methods and provide references.

	Use		Specify Method and Reference or Link
	Yes	No	
Hand calculations and equations	( )	( )	–
Software (specify the software you use)	( )	( )	–
Other(s), specify	( )	( )	

8) How do you estimate the peak discharge and the associated water depth corresponding to a flood recurrence interval for gaged and non-gaged sites? Please provide links to the relevant sources and comments.

	Use		Instrumentation in Sites		Comments and Links to Source
	Yes	No	Gaged	Non-Gaged	
Direct available data	( )	( )	[ ]	[ ]	–
Regression equation	( )	( )	[ ]	[ ]	–
Software (please specify the software and the models you use)	( )	( )	[ ]	[ ]	–
Other(s), specify	( )	( )	[ ]	[ ]	–

9) When overtopping takes place, how long does it typically last?

	Riverine Flow	Riverine Project Name and Contact Information	Coastal Flow	Coastal Project Name and Contact Information
	Riverine		Coastal Flow	
0.5 hour	[ ]	–	[ ]	–
1 hour	[ ]	–	[ ]	–
2 hours	[ ]	–	[ ]	–
5 hours	[ ]	–	[ ]	–
10 hours	[ ]	–	[ ]	–
24 hours	[ ]	–	[ ]	–
48 hours	[ ]	–	[ ]	–
Other(s), specify	[ ]	–	[ ]	–

PART VI: GEOTECHNICAL CONSIDERATIONS

10) What is the soil type typically used to build highway embankments?

	Use		Comments
	Yes	No	
Homogeneous Embankment: Coarse Grained Soils (sand and gravel)	( )	( )	–
Homogeneous Embankment: Fine Grained Soils (silt and clay)	( )	( )	–
Zoned Embankment	( )	( )	–
Other materials (specify)			–

11) Please indicate the in-situ tests and/or laboratory tests carried out to obtain the design parameters used in the design of highway embankments subjected to flooding (e.g., erosion tests, strength tests, classification tests).

	Test Name
1	–
2	–
3	–
4	–
5	–
6	–

PART VII: DESIGN AND CONSTRUCTION CONSIDERATIONS

12) Please specify the chosen design life of embankments.

	Use		Comments
	Yes	No	
5 years	( )	( )	–
10 years	( )	( )	–
25 years	( )	( )	–
50 years	( )	( )	–
75 years	( )	( )	–
100 years	( )	( )	–
Other(s), specify	( )	( )	–
None	( )	( )	–

13) Are there any special pavement or subbase requirements/techniques that are used to decrease the chance of pavement loss due to flooding?

Please specify:

( ) Yes

( ) No

If Yes, please specify the method:

1: ______________________________

2: ______________________________

3: ______________________________

4: ______________________________

5: ______________________________

Please provide references:

1: ______________________________

2: ______________________________

3: ______________________________

4: ______________________________

5: ______________________________

PART VIII: PROTECTION, MAINTENANCE, AND CONTROL TECHNIQUES

14) What current design, maintenance, and repair techniques do you use to minimize roadway embankment damage during flooding? Please check all that apply and specify an approximate cost for each selected method.

	Use		Cost ($/square feet of cover)			Time required for Installation (days/100 feet of embankment length)		
	Yes	No	High	Intermediate	Low	Long	Intermediate	Short
Precast concrete blocks (cable-tied articulated block systems, interlocking, overlapping)	( )	( )	( )	( )	( )	( )	( )	( )
Armoring (roller- compacted concrete, reinforced concrete)	( )	( )	( )	( )	( )	( )	( )	( )
Vegetation and turf reinforcement	( )	( )	( )	( )	( )	( )	( )	( )
Riprap	( )	( )	( )	( )	( )	( )	( )	( )
Gabions	( )	( )	( )	( )	( )	( )	( )	( )
Geotextile	( )	( )	( )	( )	( )	( )	( )	( )
Other(s), please specify under the "Comments" section	( )	( )	( )	( )	( )	( )	( )	( )

Comments: ____

15) What type of vegetation, vegetation cover density, and height of the vegetation is used to protect highway embankments from damage during flooding? Please provide a link to the relevant criteria used by your agency and any comments under the "Comments" section.

	Use		Density			Height		
	Yes	No	Sparse/ Scattered (10%–50%)	Continuous Light (50%– 80%)	Continuous Dense (90%–100%)	Tall (>2ft)	Medium (6–12 in.)	Short/ Cut (<6 in.)
Weeds	( )	( )	( )	( )	( )	( )	( )	( )
Brush	( )	( )	( )	( )	( )	( )	( )	( )
Willows	( )	( )	( )	( )	( )	( )	( )	( )
Trees/Tree Stumps	( )	( )	( )	( )	( )	( )	( )	( )
Pasture Grass	( )	( )	( )	( )	( )	( )	( )	( )
Bermuda Grass	( )	( )	( )	( )	( )	( )	( )	( )
Weeping Lovegrass	( )	( )	( )	( )	( )	( )	( )	( )
Other(s), specify under the "Comments" section	( )	( )	( )	( )	( )	( )	( )	( )

Comments: ____

16) Sedimentation in rivers raises the river bottom and increases the chance of flooding. What sediment control techniques do you use upstream of highway embankments to capture sediments and minimize the potential for flooding? Please provide relevant references and any comments under the "Comments" section.

	Use	
	Yes	No
Bales	( )	( )
Silt Fencing	( )	( )
Retaining Walls	( )	( )
Filter Strips	( )	( )
Other(s), specify under the "Comments" section	( )	( )

Comments: ____

PART IX: PROBABILITY AND RISK

17) What probability of failure does your agency consider acceptable for a highway embankment and for a pavement? Please specify in the "Comments" section how this probability is selected.

	Probability of Failure							Comments
	1/100,000	1/10,000	1/1,000	1/100	1/10	Differs with project, elaborate:	Other(s), specify	
Embankment	( )	( )	( )	( )	( )	( )	( )	–
Pavement	( )	( )	( )	( )	( )	( )	( )	–

18) Does your agency tie the selection of the probability of failure to the value of consequences?

( ) Yes

( ) No

Comments: ____________________

PART X: DECISION-MAKING PROCESS AND FUNDING

19) Who is responsible for repairing highway embankment damage due to flooding in your state? Please specify the agency.

[ ] 1: ____________________

[ ] 2: ____________________

[ ] 3: ____________________

[ ] 4: ____________________

[ ] 5: ____________________

Comments: ____________________

20) Please specify the source of funding available in cases of emergency repair due to damage by flooding events. Please elaborate on your preferred source and/or the advantages/limitations of the selected source under the "Comments" section. Also provide hyperlinks to funding processes.

[ ] ER Funding (Emergency Relief Funding)

[ ] FEMA (Federal Emergency Management Agency)

[ ] Other(s), specify: ____________________

Comments: ____________________

21) Sometimes, there may be a request by a related agency to design the embankment as a levee and this impacts the design or repair process. What are the factors that influence the design and/ or repair decision process to minimize highway embankment damage during flooding?

[ ] Funding agency (e.g., FEMA, ER funding); specify the name of the agency:

____

[ ] Requests of other agencies; specify the name of the agency:

____

[ ] Time considerations:

____

[ ] Feasibility considerations:

____

[ ] Prioritizing between two or more embankment projects:

____

[ ] Environmental considerations; specify the name of the agency:

____

[ ] Request that embankment be designed as a levee:

____

[ ] Other(s), specify:

____

Comments: ______________________________

PART XI: CURRENT AND FUTURE RESEARCH AREAS

22) What research do you know of that has been conducted to help minimize highway embankment damage due to flooding?

	Organization	Contact Person	Contact Information/ Hyperlink
		–	–
1		–	–
2	–	–	–
3	–	–	–
4	–	–	–
5	–	–	–
6	–	–	–

23) Which areas of embankment damage due to flooding require further research? Please prioritize (from most important -1- to least important -6-).

1: ______________________________

2: ______________________________

3: ______________________________

4: ______________________________

5: ______________________________

6: ______________________________

Thank You Page

Thank You!

Thank you for taking our survey. Your response is very important to us. If you have any questions or comments, please feel free to contact Professor Jean-Louis Briaud (contact information provided below).

Jean-Louis BRIAUD, PhD
Professor and Holder of the Buchanan Chair
Zachry Dept. of Civil Engineering
Texas A&M University
College Station, TX 77843-3136, USA
Tel: 979-845-3795
Cell: 979-777-1692
E-mail: briaud@tamu.edu

Abbreviations and acronyms used without definitions in TRB publications:

A4A	Airlines for America
AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FAST	Fixing America's Surface Transportation Act (2015)
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
MAP-21	Moving Ahead for Progress in the 21st Century Act (2012)
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TDC	Transit Development Corporation
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation

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