BERM FAILURE TECHNICAL ANALYSES

SOUTH PLATTE GRAVEL PIT EVALUATION CRITERIA

Prepared for
Urban Drainage and
Flood Control District

Wright Water Engineers, Inc.

January 2013
121-030.000

TABLE OF CONTENTS

TABLES

FIGURES

ATTACHMENTS

Berm Failure Technical Analyses
South Platte Gravel Pit Evaluation Criteria

INTRODUCTION

The 1987 *Technical Review Guidelines for Gravel Mining Activities Within or Adjacent to 100-Year Floodplains* (Guidelines) is based upon the 1985 criteria developed for Adams County. The 1985 and 1987 parameters as published represent the combined knowledge and experience of the many dozens of individuals who conferred on original and early Wright Water Engineers, Inc. (WWE) draft documents and criteria.

The criteria of the guidelines were based upon engineering principles and practical objectives related to rock product mining of the 1980s. Original and initial guideline drafts in early 1985 were modified via significant industry and regulatory agency input during the spring of 1985. The technical criteria given in the 1987 guidelines are based upon principles, policy and criteria from the 1969 Urban Drainage and Flood Control District (UDFCD) Urban Storm Drainage Criteria Manual (USDCM), the 1984-1985 Master Plan, industry practices and economics, regulatory agency needs and the civil and geotechnical engineering professions. The parameters are supportable.

In 2012, WWE prepared updates to the guidelines, including evaluation of the original setback requirements using research performed and calculation methods developed over the 25-years since the guidelines were first published. The calculation methods are discussed in greater detail in the attachments to this memorandum. The calculation attachments are as follows:

Attachment A–Embankment Overtopping Analysis per Federal Highway Administration (FHWA) Method

Attachment B–Dam Breach Analysis per Colorado State Engineer's Office (SEO) Guidance

Attachment C–Steep Slope Riprap Sizing per the Gravel Guidelines

Attachment D–Steep Slope Riprap Sizing per Hydrologic Engineering Circular (HEC) 23

ASSUMPTIONS

For Attachments A through D, the following assumptions were used for each set of stability test calculations:

- The embankment overtopping depth varies between 2-feet and 6-feet.
- Overtopping flows were bracketed between 1,000 cfs and 20,000 cfs.
- For unprotected or natural vegetation berms, an assumed top width of 300-feet was analyzed.
- The downstream gravel pit is essentially empty. A similar hydraulic drop is assumed for overflow to the River.
- The flooding event will last 24 to 48 hours.

BERM FAILURE

The Attachment A and B calculations were performed for comparison with each other. The attachment A calculation method was developed through a series of large-scale hydraulic model experiments to simulate floods overtopping highway embankments. The hydraulic model experiments varied several parameters, including crest cover with pavement, grass, or bare soil, and embankment slopes either covered with grass of bare soil. These conditions are similar to the gravel pit berms in that the roadway embankment soil compaction and soil type may vary, as is the possible case with gravel pit berms which consist of natural undisturbed soil or overburden placed at along the edge of the pit excavation during reclamation efforts. The study referenced in attachment A was issued in March of 1987. The Attachment B dam breach analysis was used for comparison with the attachment A results. The dam breach guidelines were assembled by the Colorado SEO Dam Safety branch to provide guidance for dam failure inundation mapping and assigning dam hazard classifications. This guidance documents was released in February of 2010 and summarized numerous methods of dam breach modeling based on regression equations developed from data bases of actual dam failures. The dam breach guidance is not a direct comparison because dams tend to be constructed with a greater amount of soil compaction and compaction testing than roadway embankments and the dam breach analysis focuses on peak flow rate and breach size once the dam failure begins, rather than a prolonged overtopping scenario. The dam breach analysis does provide a reasonableness check for the roadway embankment failure analysis as well as better representing the gravel berm heights of nearly 30-feet when compared with the typical roadway embankment height of up to 15-feet. Figure 1 shows the dimensions and variables of the example berm analyzed.

The results of the calculations presented in attachments A and B are summarized below for comparison purposes.

Table 1
Volume of Eroded Embankment
Lateral Berms without Protection

Condition		Erosion Volume per Foot of Embankment Length		Percent of Embankment Eroded
		CY/FT	CF/FT	
30-Foot Tall Berm	Bare Soil, 6 Feet Overtopping for 24 Hours	323	8,709	78 %
	Bare Soil, 6 Feet Overtopping for 48 Hours	605	16,330	100%
	Paved Crest, 6 Feet Overtopping for 24 Hours	271	7,309	65%
15-Foot Tall Berm	Bare Soil, 6 Feet Overtopping for 24 Hours	155	4,180	82%
	Bare Soil, 6 Feet Overtopping for 48 Hours	290	7,838	100%
	Paved Crest, 6 Feet Overtopping for 24 Hours	130	3,509	69%
30-Foot Tall Berm	Bare Soil, 2 Feet Overtopping for 24 Hours	169	4,572	40%
	Bare Soil, 2 Feet Overtopping for 48 Hours	318	8,573	76%
	Paved Crest, 2 Feet Overtopping for 24 Hours	161	4,355	39%

Note: The 30-foot tall berm has a volume of 417 cy/ft and the 15-foot tall berm has a volume of 188 cy/ft.

Table 2
Dam Breach Analysis—Volume of Eroded Soil, Peak Flow Rate from Breach and Time for Breach Formation

Calculation Method		Peak Breach Discharge	Breach Formation Time	Volume of Material Eroded	Avg. Breach Width	Erosion Volume per Foot of Embankment Length		Percent of Embankment Eroded[4]
		(cfs)	(hours)	(cy)	(ft)	(cy/ft)	(cf/ft)	(%)
30-Foot Tall Berm	MLM Method [1] 6 feet Overtopping	10,087	0.55	10,126	40	254	6,860	61
	Froehlich Method[2] 6 feet Overtopping, Overtopping Failure	13,981	0.71	48,000	116	414	11,170	99
	Froehlich Method[2] 6 feet Overtopping, Piping Failure	14,764	0.71	37, 000	89	417	11,250	100
15-Foot Tall Berm	MLM Method 6 feet Overtopping	8,530	0.52	8,590	46	186	5,040	100
	Froehlich Method[3] 6 feet Overtopping, Overtopping Failure	3,411	1.43	21,150	112	189	5,100	100
	Froehlich Method[3] 6 feet Overtopping, Piping Failure	3,890	1.43	16,250	86	189	5,100	100
30-Foot Tall Berm	MLM Method [1] 2 feet Overtopping	6,952	051	8,274	36	230	6,205	55
	Froehlich Method[2] 2 feet Overtopping, Overtopping Failure	12,635	071	48,000	116	414	11,170	100
	Froehlich Method[2] 2 feet Overtopping, Piping Failure	13,196	0.71	37,000	89	417	11,250	100

[1] Berm height less than listed height used due to calculation method limits.
[2] Eroded volume based on an average berm width of 375 feet.
[3] Eroded volume based on an average berm width of 340 feet.
[4] Percent of embankment eroded assumes consistent erosion along entire breach width, while the calculation method assumes a full breach through the dam occurs.

The results above indicate a significant portion of the berm could be eroded during a flood event. The calculations above only account for one failure method at a time; however, animal burrow holes in the embankment can lead to a piping type failure after partial failure has occurred due to overtopping.

Comparing the results of Attachment A and B shows an order of magnitude similarity in the estimated volume of soil eroded. The difference between the results can be attributed to the fact that the calculations are based on different assumptions. The dam breach calculations are intended to predict the breach size and time to failure under the assumption that a breach will occur; while the embankment overtopping calculations are intended to show the volume of earth moved during an overtopping event which might not cause complete failure of the embankment. It is worth noting that the dam breach calculations indicate the full breach will form within 30 minutes to 1.5 hours from the time the breach begins. This shows that if a breach begins to form in the gravel pit berm, it will quickly open and allow water to flow downstream, which can result in more failures downstream due to the in rush of water. The flow rate associated with a breach is a result of the potential energy in the stored water; the flow is above the flow rate occurring in the river. In addition, the sudden draining of a gravel pit can cause instability in the upstream banks which could cause the next upstream gravel pit to breach its bank.

RIPRAP SIZING

The Attachments C and D present calculations for flow down riprap on steep slopes, in particular for protecting the pitside banks during overtopping from the river. These calculations are also applicable for overtopping flow from one pit to the next down river pit or return flow from the pit to the river. For both attachments, flow over the embankment crest and down the steep slope was modeled as a wide modified trapezoidal channel with a 100-foot bottom width and sideslopes of 0.1 percent (1,000 H: 1V). Calculations were based on embankment overflows of 1,000 cfs and 20,000 cfs to bracket the range of potential overtopping flow along the South Platte River.

Attachment C uses the pitside bank steep riprap slope sizing method presented in the 1987 guidelines (Section 2.4.1). The method and equations are similar to the method presented in HEC 11, which was issued in 1989 and revised in 2000 with metric units.

Attachment D calculations are based on a steep slope method presented in HEC 23, Design Guidance 5. HEC 23 is a large, two volume publication covering bridge scour and stream instability by the FHWA. HEC 23, which was released in 2009, addresses roadway overtopping and relies on the FHA document referenced in Attachment A. The method in Attachment D was developed in the 1990's at Colorado State University Hydraulics Laboratory Flume.

In Attachments C and D, each of the methods reviewed indicate a very thick layer of large riprap (larger than the UDFCD maximum size VH/D_{50} = 24") is necessary on steep (2½: 1) slopes for high flows. The results of the two methods are summarized for comparison in Table 3 below. These results, which are not practical given local material availability and the cost impact of lining long berms, support alternate methods of bank stabilization. For example, the use of spillways will concentrate flow in specific areas, allowing heavy armoring to be placed selectively for high overtopping flow rates. This allows the design flow rate for the remainder of the pitside bank to be reduced, which would allow smaller riprap or boulders. Other methods to avoid large riprap or boulders are grouted boulders, soil cement or flatter side slopes.

Table 3
Riprap Sizing Comparison

Bank Slope	UDFCD Gravel Guidelines Method		HEC Steepslope Method[1]	
	1,000 cfs	20,000 cfs	1,000 cfs	20,000 cfs
2.5H:1V	D_{50}=3.5 ft	D_{50}=8.9 ft	D_{50}=1.0 ft t=2.0 ft	D_{50}=6 ft t=12 ft
3H:1V	D_{50}=2.2 ft	D_{50}=6.8 ft	D_{50}=1.0 ft t=2.0 ft	D_{50}=6 ft t=12 ft
4H:1V	D_{50}=1.4 ft	D_{50}=4.3 ft	D_{50}=1.0 ft t=2.0 ft	D_{50}=6.5 ft t=13 ft
5H:1V	D_{50}=1.1 ft	D_{50}=3.2 ft	N/A	N/A

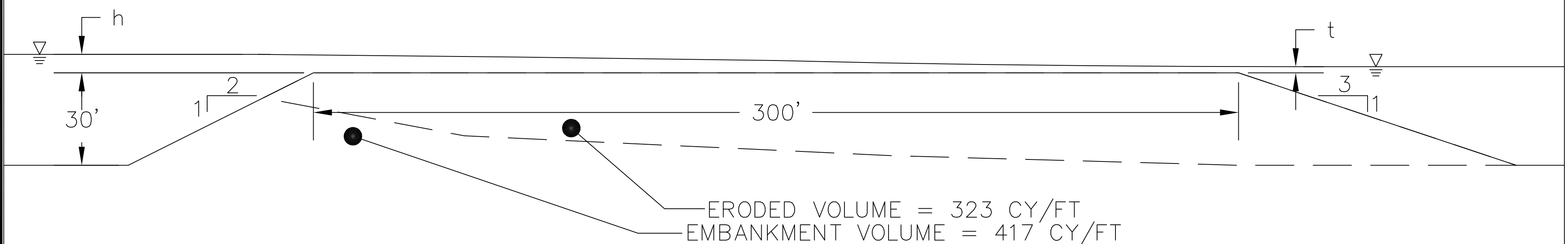
[1] HEC Steepslope method does not include a safety factor.

The calculations do indicate that in lower flow conditions (1,000 cfs) a smaller size riprap could be used on steep slopes under the Attachment D calculation method than the Attachment C method, but it is worth noting that the method used in the calculation is based on a riprap gradation that is locally non-standard (in the Denver metro area). In addition, the Attachment D method was developed based on flow tests using a 3-meter wide test flume in the 1990's and has not been subject to years of experience like the UDFCD method and gradations have been. Based on these factors, the riprap sizing method contained in the Guidelines still is reasonable to protect a resource as important at the South Platte River.

The existing 1987 guidelines did not offer specific direction for selecting the design flow for the overtopping protection due to the variable geometries available along the river; the intent was and remains to allow the design engineer to determine a reasonably conservative flow rate as appropriate for the local river reach. We would suggest the minimum flow should be the overbank flow during the 100-year event, based on the modeling results presented in the Master Plan or Flood Hazard Area Delineation report.

Z:\Project Files\12\121-030\121-030.000\Deliverables\gravel tech basis memo 11-6-12.docx

Figure



WWE
WRIGHT WATER
ENGINEERS, INC.
2490 W. 26TH AVE. 100A
DENVER, CO 80211
(303)480-1700

DESIGN –
DETAIL JMN
CHECK KRW
DATE 11/20/12
SCALE 1"=50'

FIGURE 1

EMBANKMENT CROSS SECTION
EXAMPLE LATERAL BERM

Attachment A
Embankment Overtopping Analysis per Federal Highway Administration (FHWA) Method

Wright Water Engineers, Inc.

CALCULATION SHEET

Project: UDFCD Gravel Mining Criteria
Job. No.: 121-030.000
Date: 9/11/2012
Subject: Lateral Berm Erosion Analysis

Design: JMN
Check: TAE

I. Purpose

Estimate volume of material eroded from embankments during overtopping

II. References

1. Development of a Methodology for Estimating Embankment Damage Due to Flood Overtopping, FHA Report No FHWA/RD-86/126, March 1987
2. FHAD South Platte River, Adams County, CO., UDFCD/CDM April 2005
3. Technical Review Guidelines for Gravel Mining Activities Within or Adjacent to 100-Year Floodplains, UDFCD/WWE Dec. 1987 (April, 2004)

III. Assumptions

1. Embankment overtopping occurs for 24 fours to 48 hours.
2. Noncohessive soils
3. Average Erosion Rate: $E = K_1K_2E_a$ Ref # 1, Eqn 33
   where:
   K_1 = time adjustment factor for flood duration
   K_2 = adjustment factor for embankment height
   E_a = erosion rate for a 5-foot embankment
   E_a is based on the ratio of tailwater to headwater (t/h)
4. Erosion Volume V_s =E * time (hours)
5. For purposes of these calculations the supply of sediment from embankments is not a limiting factor.

IV. Calculations

1. Case 1, 300 foot wide berm, 3:1 slope downstream, 2:1 slope upstream and bare soil; 24 hrs

t =	-3				
h =	6		also equals overtopping depth		
t/h =	-0.5		if t/h <= 0, use free fall curves		
duration =	24	hours			
K_1 =	0.64		Ref #1, Fig 57		ok
30-foot tall embankment					
K_2 =	7.5		Ref #1, Fig 58 (projected)		ok
Bare Soil Embankment					
E_a =	2.8		Ref #1, Fig 47 (projected)		ok
E =	13.4	cy/hr/ft			ok
duration =	24	hours			
V_s =	**323**	**cy/ft**	**=**	**8709 cf/ft**	ok

2. Case 2, 300 foot wide berm, 3:1 slope downstream, 2:1 slope upstream and bare soil; **48 hrs**

t =	-3	
h =	6	also equals overtopping depth
t/h =	-0.5	if t/h <= 0, use free fall curves

duration =	48	hours				
K_1 =	0.6		Ref #1, Fig 57			ok
30-foot tall embankment						
K_2 =	7.5		Ref #1, Fig 58 (projected)			ok
Bare Soil Embankment						
E_a =	2.8		Ref #1, Fig 47 (projected)			ok
E =	12.6	cy/hr/ft				
duration =	48	hours				
V_s =	**605**	**cy/ft**	**=**	**16330**	**cf/ft**	ok

3. Case 3, 300 foot wide berm, 3:1 slope downstream, 2:1 slope upstream and **paved top** and bare slopes; **24 hrs**

t =	-3					
h =	6		also equals overtopping depth			
t/h =	-0.5		if t/h <= 0, use free fall curves			
duration =	24	hours				
K_1 =	0.64		Ref #1, Fig 57			ok
30-foot tall embankment						
K_2 =	7.5		Ref #1, Fig 58 (projected)			ok
Paved Top, Bare Slopes Embankment						
E_a =	2.35		Ref #1, Fig 51			ok
E =	11.3	cy/hr/ft				
duration =	24	hours				
V_s =	**271**	**cy/ft**	**=**	**7309**	**cf/ft**	ok

Test cases 1-3 with embankment height equal to the maximum height publised in reference 1 (15-feet).

1a. Case 1a, 300 foot wide berm, 3:1 slope downstream, 2:1 slope upstream and bare soil; 24 hrs

t =	-3					
h =	6		also equals overtopping depth			
t/h =	-0.5		if t/h <= 0, use free fall curves			
duration =	24	hours				
K_1 =	0.64		Ref #1, Fig 57			ok
15-foot tall embankment						
K_2 =	3.6		Ref #1, Fig 58			ok
Bare Soil Embankment						
E_a =	2.8		Ref #1, Fig 47 (projected)			ok
E =	6.5	cy/hr/ft				
duration =	24	hours				
V_s =	**155**	**cy/ft**	**=**	**4180**	**cf/ft**	ok

2a. Case 2a, 300 foot wide berm, 3:1 slope downstream, 2:1 slope upstream and bare soil; 48 hrs

t =	-3					
h =	6		also equals overtopping depth			
t/h =	-0.5		if t/h <= 0, use free fall curves			
duration =	48	hours				
K_1 =	0.6		Ref #1, Fig 57			ok
15-foot tall embankment						
K_2 =	3.6		Ref #1, Fig 58			ok
Bare Soil Embankment						
E_a =	2.8		Ref #1, Fig 47 (projected)			ok
E =	6.0	cy/hr/ft				ok
duration =	48	hours				
V_s =	**290**	**cy/ft**	**=**	**7838**	**cf/ft**	ok

3a. Case 3a, 300 foot wide berm, 3:1 slope downstream, 2:1 slope upstream and paved top and bare slopes; 24 hrs

t =	-3		
h =	6		also equals overtopping depth

t/h =	-0.5	if t/h <= 0, use free fall curves			
duration =	24 hours				
K_1 =	0.64	Ref #1, Fig 57			ok
15-foot tall embankment					
K_2 =	3.6	Ref #1, Fig 58			ok
Paved Top, Bare Slopes Embankment					
E_a =	2.35	Ref #1, Fig 51			ok
E =	5.4 cy/hr/ft				ok
duration =	24 hours				
V_s =	**130 cy/ft**	**=**	**3509**	**cf/ft**	ok

Rerun cases 1-3 with overtopping height equal to 2-feet to establish a range of erosion volumes.

4. Case 4, 300 foot wide berm, 3:1 slope downstream, 2:1 slope upstream and bare soil; 24 hrs

t =	-3				
h =	**2**	also equals overtopping depth			
t/h =	-1.5	if t/h <= 0, use free fall curves			
duration =	24 hours				
K_1 =	0.64	Ref #1, Fig 57			
30-foot tall embankment					
K_2 =	7.5	Ref #1, Fig 58 (projected)			
Bare Soil Embankment					
E_a =	**1.47**	Ref #1, Fig 47			
E =	7.1 cy/hr/ft				
duration =	24 hours				
V_s =	**169 cy/ft**	**=**	**4572**	**cf/ft**	ok

5. Case 5, 300 foot wide berm, 3:1 slope downstream, 2:1 slope upstream and bare soil; **48 hrs**

t =	-3				
h =	**2**	also equals overtopping depth			
t/h =	-1.5	if t/h <= 0, use free fall curves			
duration =	48 hours				
K_1 =	0.6	Ref #1, Fig 57			
30-foot tall embankment					
K_2 =	7.5	Ref #1, Fig 58 (projected)			
Bare Soil Embankment					
E_a =	**1.47**	Ref #1, Fig 47			
E =	6.6 cy/hr/ft				
duration =	48 hours				
V_s =	**318 cy/ft**	**=**	**8573**	**cf/ft**	ok

6. Case 6, 300 foot wide berm, 3:1 slope downstream, 2:1 slope upstream and **paved top** and bare slopes; **24 hrs**

t =	-3				
h =	**2**	also equals overtopping depth			
t/h =	-1.5	if t/h <= 0, use free fall curves			
duration =	24 hours				
K_1 =	0.64	Ref #1, Fig 57			
30-foot tall embankment					
K_2 =	7.5	Ref #1, Fig 58 (projected)			
Paved Top, Bare Slopes Embankment					
E_a =	**1.4**	Ref #1, Fig 51			
E =	6.7 cy/hr/ft				
duration =	24 hours				
V_s =	**161 cy/ft**	**=**	**4355**	**cf/ft**	ok

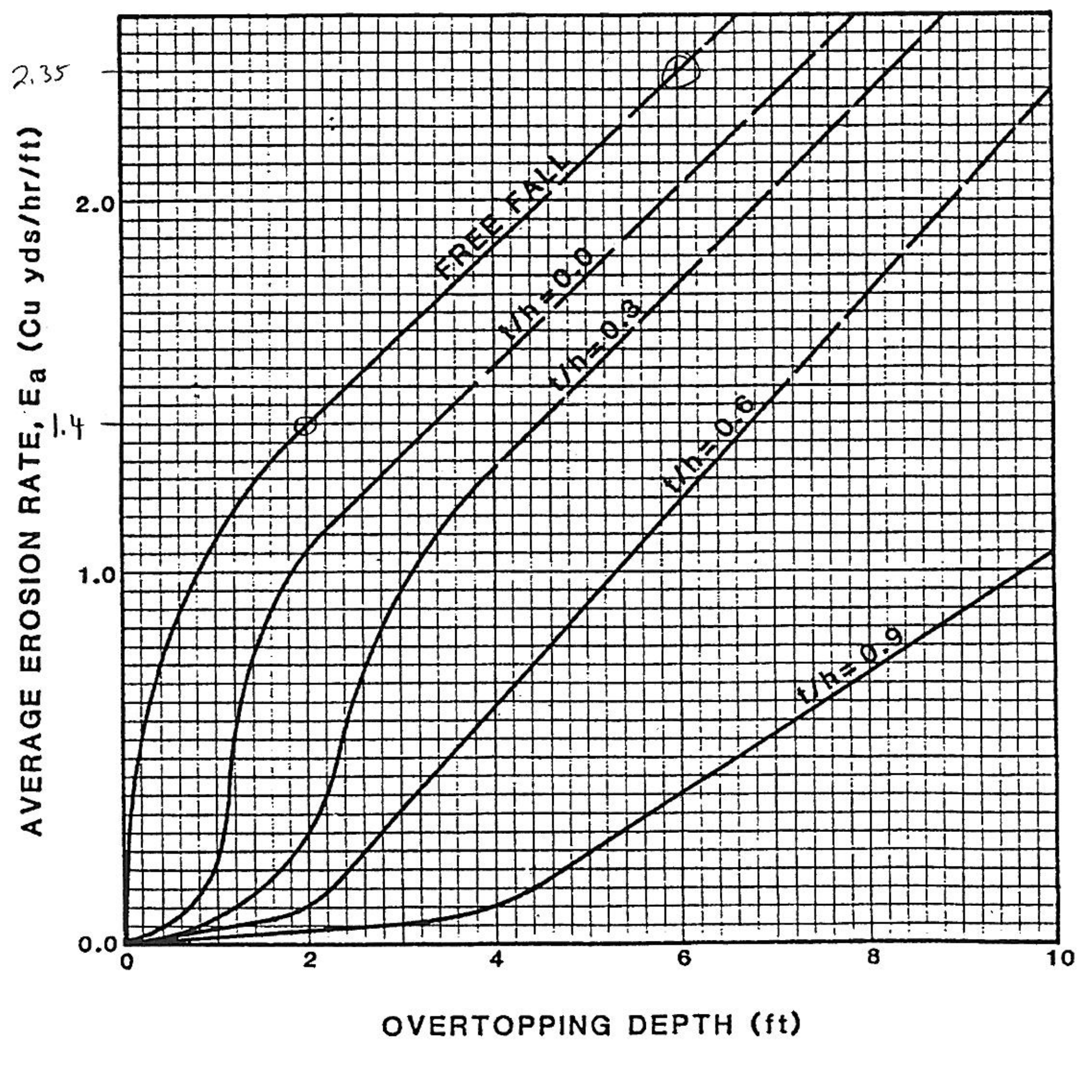


Figure 51. Average erosion rate during 4-hour flow overtopping of 5-foot paved noncohesive soil embankment without vegetal cover.

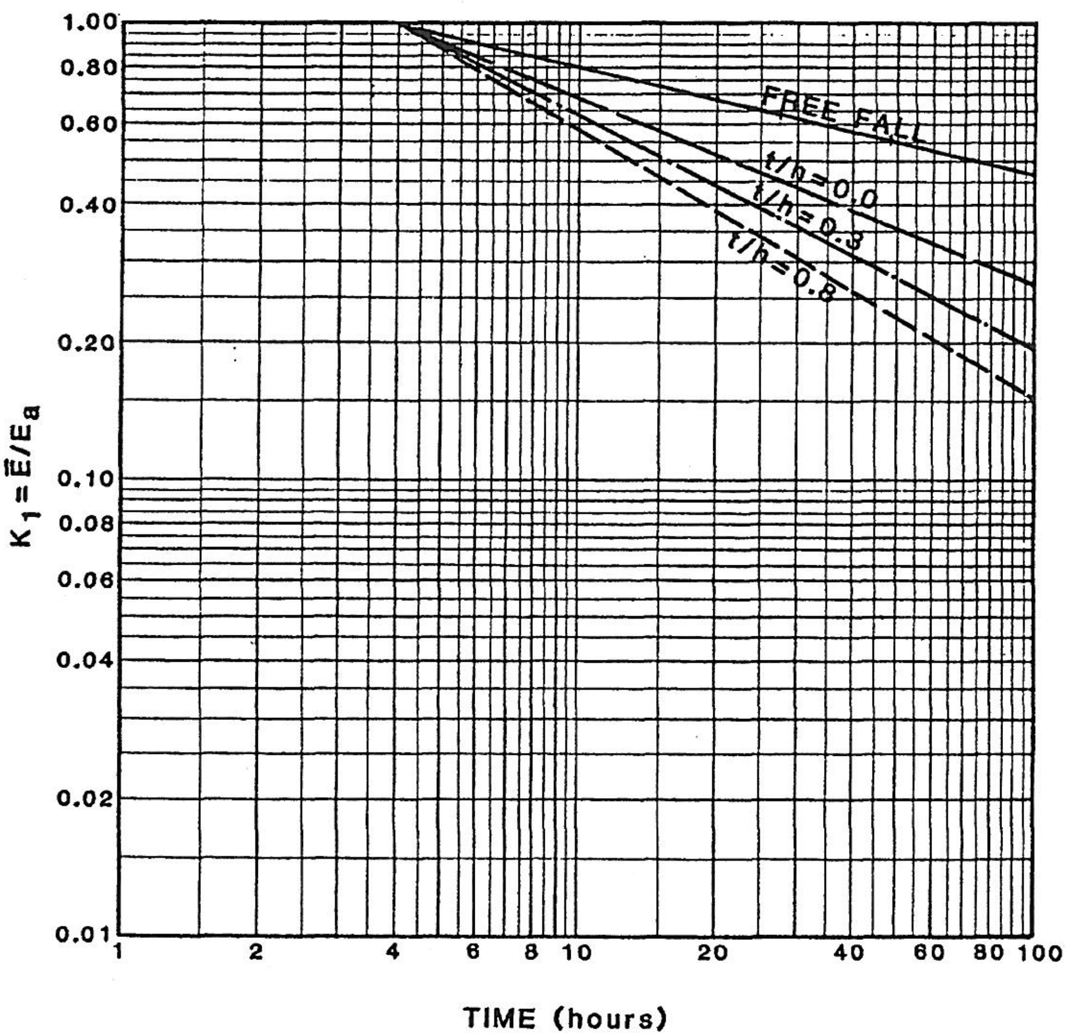


Figure 57. Average erosion rate change with time duration.

EXTRAPOLATE TO 30'
ALSO, USE 15

30ft embankment

$$K_2 = \left(\left[(30ft - 5ft) \div 5\right] * 1.3\right) + 1 = 7.5$$

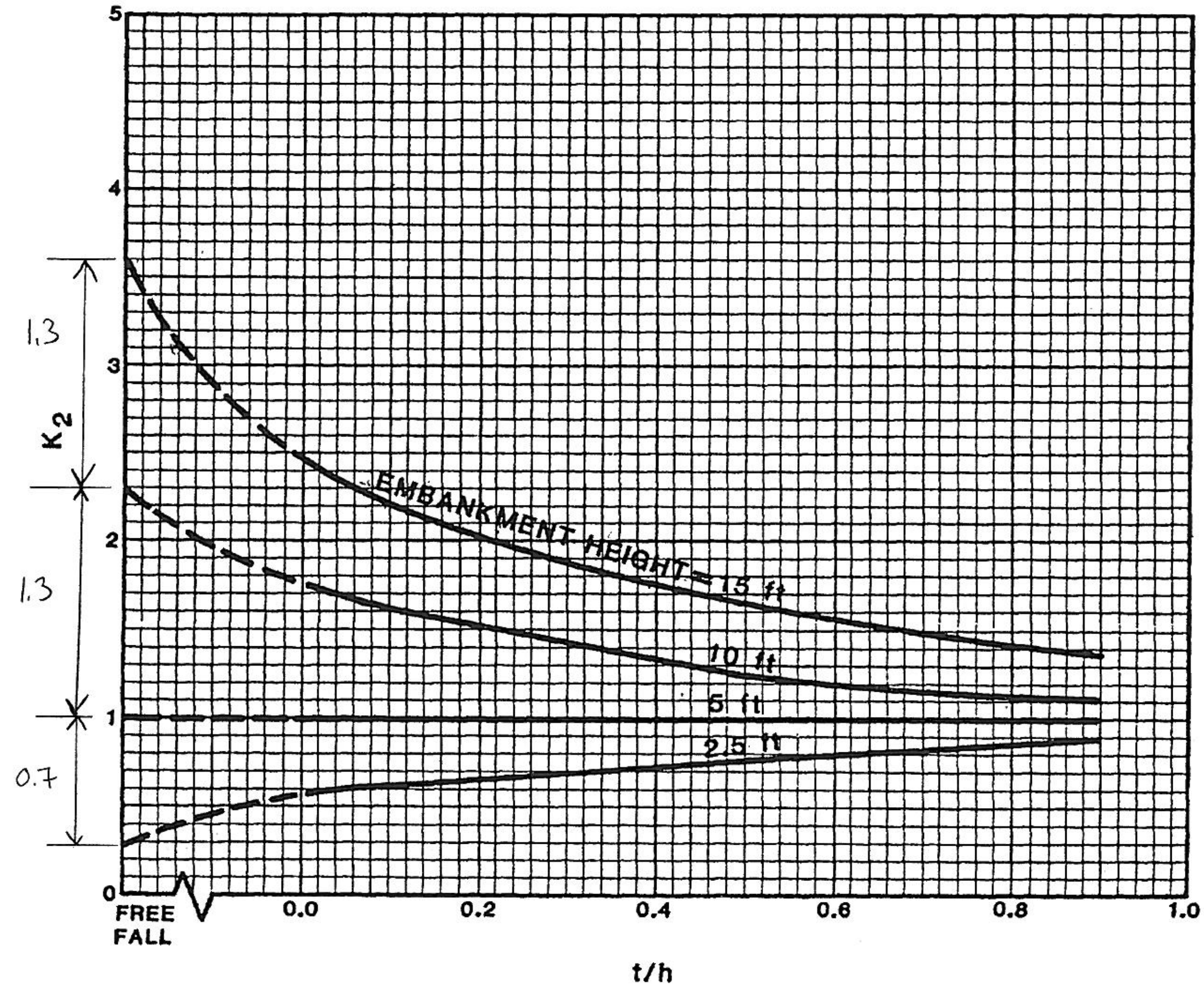


Figure 58. Adjustment factor considering embankment height.

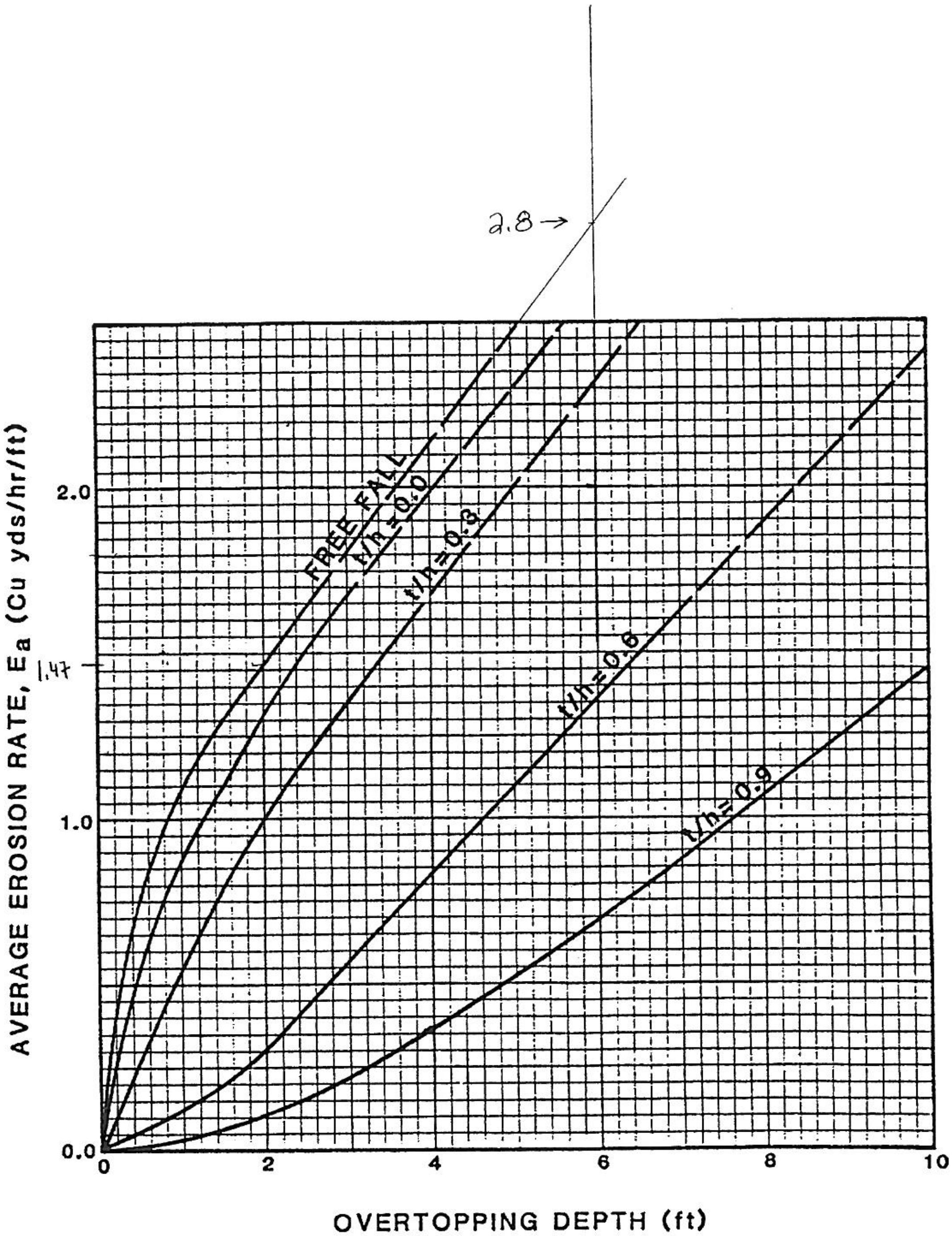


Figure 47. Average erosion rate during 4-hour overtopping of 5-foot noncohesive bare soil embankment (d_{50} less than 8 mm).

OFFICE COPY NTIS No. PB87-178356

Development of a Methodology for Estimating Embankment Damage Due to Flood Overtopping

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

$24.95

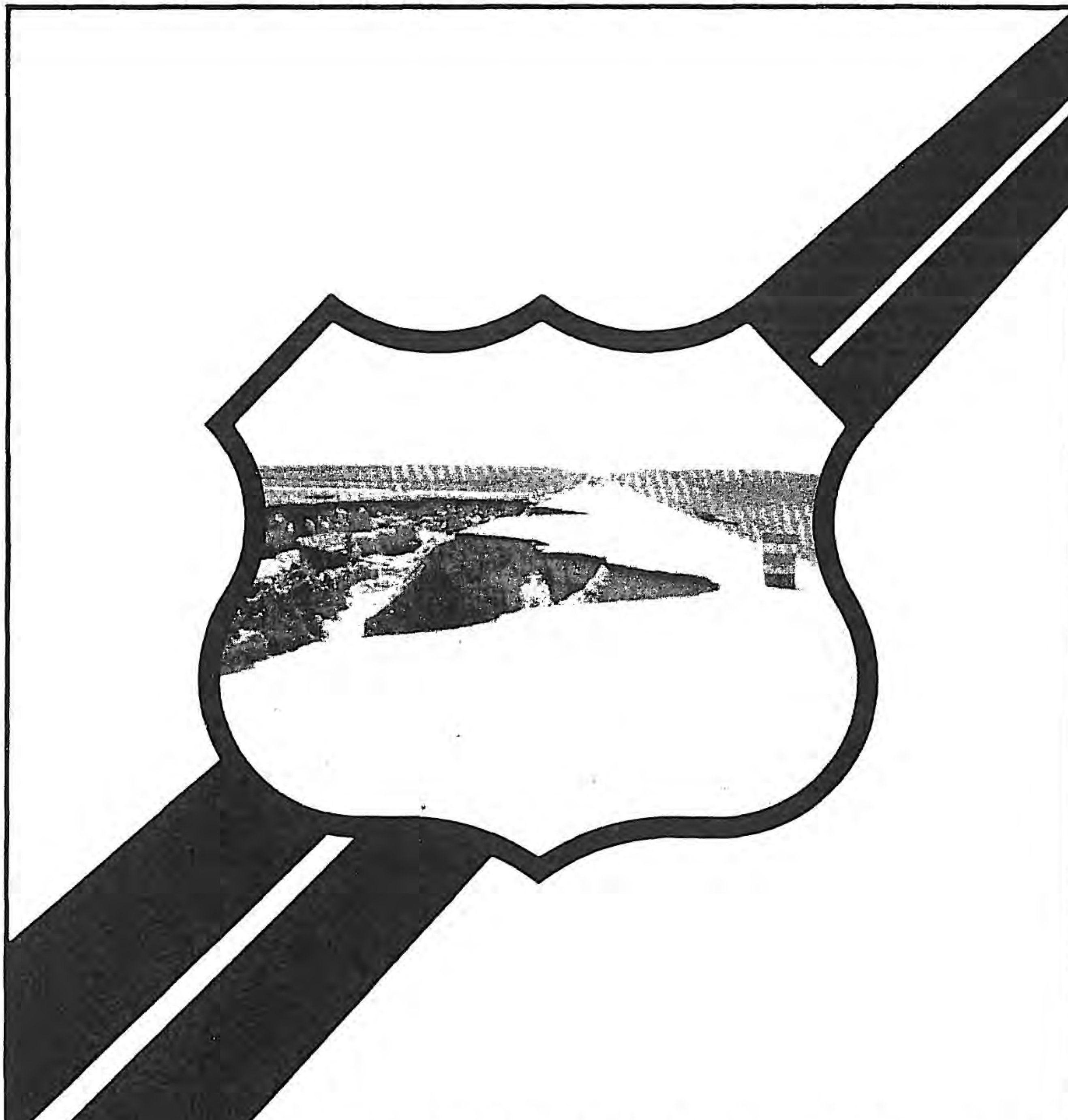
Report No.
FHWA/RD-86/126

U.S. Department of Transportation
Federal Highway Administration

Co-sponsored by:

U.S. Department of Agriculture
Forest Service
Washington, D.C. 20590

Final Report
March 1987



This document is available to the U.S. public through the National Technical Information Service, Springfield, Virginia 22161

FOREWORD

This report describes a series of large-scale hydraulic model experiments to simulate floods overtopping highway embankments. Test conditions included embankments with and without pavement, with and without grass cover, with a range of headwater and tailwater elevations, and with a limited number of protective measures. The report will be of interest to hydraulic engineers for State highway agencies, consultants and other Government agencies who deal with flood damage evaluations of highway embankments or who deal with evaluations of dam safety in general.

Sufficient copies of the report are being distributed to provide a minimum of two copies to each FHWA regional office, one copy to each FHWA division office and one copy to each State highway office. Direct distribution is being made to the division offices.

Charles F. Galambos /for
Richard E. Hay, Director
Office of Engineering and
Highway Operations
Research and Development
Federal Highway Administration

NOTICE

Technical Report Documentation Page

1. Report No.
FHWA/RD-86/126

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and Subtitle
DEVELOPMENT OF A METHODOLOGY FOR ESTIMATING EMBANKMENT DAMAGE DUE TO FLOOD OVERTOPPING

5. Report Date
March 1987

6. Performing Organization Code

7. Author(s)
Y. H. Chen and Bradley A. Anderson

8. Performing Organization Report No.

9. Performing Organization Name and Address
Simons, Li & Associates, Inc.
3555 Stanford Road
P.O. Box 1816
Fort Collins, Colorado 80522

10. Work Unit No. (TRAIS)
FCP 35H4-042

11. Contract or Grant No.
DTFH61-82-C-00104

13. Type of Report and Period Covered
Final Report
Sept. 1982 - March 1986

12. Sponsoring Agency Name and Address
Office of Engineering & Highway Operations R&D
Federal Highway Administration
6300 Georgetown Pike
McLean, Virginia 22101-2296

14. Sponsoring Agency Code

15. Supplementary Notes
FHWA Contract Manager (COTR): J. Sterling Jones (HNR-10)
Co-sponsored by:
U.S. Dept. of Agriculture
Forest Service, Washington, D.C. 20013

16. Abstract
The objectives of this study are to conduct laboratory tests and develop a methodology to quantitatively determine embankment damage and assess protective measures. During the study, available literature and field data were collected. The embankments used in this study are 6 ft (1.8 m) high, 10 to 22 ft (3.0 to 6.7 m) in crest width, and 3 ft (0.9 m) in length, with slope varying from 2:1 to 3:1. The embankment surfaces include both with and without protective measures (pavement, grass, mattresses, Geoweb, soil cement, Enkamat, and others). The flood overtopping depths ranging from 0.5 to 4 ft (0.15 to 1.22 m), discharges ranging from 1 to 25 cfs/ft (0.1 to 2.32 cms/m) and tailwater conditions ranging from 10 percent water-surface drop to free fall. A computer model was developed to determine hydraulics of overtopping flow and associated erosion damage. This model was verified using field data and laboratory test results, and was utilized to generate charts for estimating embankment damage.

17. Key Words
Embankment, Erosion, Protection, Soil, Velocity, Shear Stress, Flood Overtopping, Headwater, Tailwater, Free Fall, Mathematical Model

19. Security Classif. (of this report)

20. Security Classif. (of this page)

21. No. of Pages
215

22. Price

Form DOT F 1700.7 (8-72)

REFERENCES

(1) Simons, Li & Associates, Inc., "Presentation of Field Data on Embankment Damage Due to Flood Overtopping," Progress Report for Task A, Prepared for U.S. Department of Transportation, Federal Highway Administration, January 1984.

(2) H. W. Hjalmarson, "FLood Characteristics and Highway Damage at Five Arizona Sites, Flood of October 1983," U.S. Geological Survey prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration, Tucson, September 1984.

(3) D. L. Yarnell, and F. A. Nagler, "Flow of Flood Water Over Railway and Highway Embankments," Public Roads, Vol. II, No. 2, 30-34, 1930.

(4) C. E. Kindsvater, "Discharge Characteristics of Embankment-Shaped Weirs," Studies of Flow of Water Over Weirs and Dams, USGS Water-Supply Paper 1617-A, 1964.

(5) J. N. Bradley, "Hydraulics of Bridge Waterways," Hydraulic Design Series No. 1, U.S. Department of Transportation, Federal Highway Administration, 1973.

(6) V. T. Chow, Open-Channel Hydraulics, McGraw-Hill Book Company, New York, 1959.

(7) J. N. Bradley, and A. J. Peterka, "Hydraulic Design of Stilling Basins: Stilling Basin with Sloping Apron (Basin V), Journal of Hydraulics Division, ASCE, Vol.83, No. HY5, October 1957.

(8) F. C. Townsend, and D. J. Goodings, "Centrifugal Model Analysis of Coal Waste Embankment Stability," U.S. Army Engineer Waterways Experiment Station for Bureau of Mines, Open File Report, 1979.

(9) P. Miller, Personal Communication, U.S. Army Engineer Waterways Experiment Station, 1983.

(10) E. R. Tinney, and H. Y. Hsu, "Mechanics of Washout of an Erodible Fuse Plug," Trans. ASCE, Paper No. 3283, Vol. 127, Part 1, 1962, pp. 31-59. Library Reference.

(11) J. Gessler, "Beginning and Ceasing of Sediment Motion," River Mechanics, Chapter 7, edited by H.W. Shen, 1971.

(12) S. Fortier and F. C. Scobey, "Permissible Canal Velocities," Trans. ASCE, Vol. 89, 1926, pp. 940-956.

(13) E. T. Smerdon, and R. P. Beasley, "Relation of Compaction and Other Soil Properties to Erosion Resistance of Soils," Trans. ASCE, Vol. 8, 1959.

REFERENCES (continued)

(14) E. T. Smerdon, and R. P. Beasley, "The Tractive Force Theory Applied to Stability of Open Channels in Cohesive Soils," Research Bulletin 715, University of Missouri, College of Agriculture, Agr. Exp. Station, October, 1959.

(15) E. H. Grissinger, "Resistance of Selected Clay Systems to Erosion by Water," Water Resources Research, Vol. 2, No. 1, 1966, pp. 131-138.

(16) W. Lyle, and E. Smerdon, "Relation of Compaction and Other Soil Properties to the Erosion Resistance of Soils," Trans. ASAE, 1965.

(17) A. Arumugam, "Fundamental Aspect of Surface Erosion of Cohesive Soils," Ph.D. Dissertation, University of California, Davis, 1974.

(18) J. C. McWhorter, T. G. Carpenter, and R. N. Clark, "Erosion Control Criteria for Drainage Channels," prepared for the Mississippi State Highway Department in Cooperation with U.S. Department of Transporation, Federal Highway Administration, by Agricultural Experiment Station, Mississippi State University, State College, Mississippi, March 1968.

(19) Y. H. Chen and G. K. Cotton, "Design of Roadside Channels with Flexible Linings," Hydraulic Engineering Circular No. 15, Federal Highway Administration, February 1986.

(20) USDA Soil Conservation Service, Handbook of Channel Design for Soil and Water Conservation, 1954.

(21) J. M. Wiggert, and D. N. Contractor, "A Methodology for Estimating Embankment Failure," an unpublished paper presented to Water Resources Engineers, Inc., Springfield, VA, Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA 24060, no date, written around 1969.

(22) E. A. Cristofano, "Method of Computing Erosion Rate for Failure of Earthfill Dams," unpublished memorandum, Engineering and Research Center, Bureau of Reclamation, Department of the Interior, Denver, CO, April 1965.

(23) R. Ariathurai, and K. Arulanandan, "Erosion Rates of Cohesive Soils," Journal of Hydraulics Division, ASCE, Vol. 104, No. HY2, February 1978, pp. 279-283.

(24) S. P. Chee, "Design of Erodible Dams," in Proceedings of International Conference of Water Resources Engineerings, Asian Institute of Technology, Bankok AIT, V. 1, 105-113, 1978.

(25) V. R. Schneider, and K. V. Wilson, "Hydraulic Design of Bridges with Risk Analysis," U.S. Geological Survey for FHWA Office of Development, Report FHWA-TS-80-226, FHWA HDV-21, March 1980.

BIBLIOGRAPHY

(1) J. O. Abreu-Lima, and W. B. Morgan, "Protection of Earth Embankments by Riprap of Uniform Size," M.S. Thesis, Univ. Iowa, Iowa City, 1951.

(2) K. Arnulanandan, P. Logannathan, and R. B. Kron, "Pore and Eroding Fluid Influences on Surface Erosion of Soil," Journal of Geotechnical Engineering Division, ASCE, Vol. 101, No. GT1, January 1975, pp. 51-65.

(3) D. T. Babbit, (Chrmn), "Specialty Session of Slopes and Embankments," Proceedings of Conference on Earthquake Engineering and Soil Dynamics, ASCE, 1386, 1978.

(4) N. J. Brogdon, and V. L. Grace, "Stability of Riprap and Discharge Characteristics, Overflow Embankment, Arkansas River, Arkansas," Army Engineering Waterway Experiment Station, Vicksburg, 94 p, June 1964.

(5) J. Davidian, and D. I. Cahal, "Distribution of Shear in Rectangular Channels," Article 113, U.S. Geological Survey Professional Paper 475-C, 1963.

(6) Federal Highway Administration, "The Design of Encroachments on Flood Plains Using Risk Analysis," Hydraulic Engineering Circular No. 17, FHWA HNG-31, Washington, D.C. 20590, 1980.

(7) J. Fowler, "Design Construction and Analysis of Fabric-Reinforced Embankment Test Section of Pinto Pass, Mobile Alabama," Rpt. EL-81-7, U.S. Army Engineering Waterways Experiment Station, Vicksburg, MS; 225 p, 1981.

(8) J. Fowler, "Synthetic Fabrics for Reinforced Embankments," Civil Engineering, ASCE, 51(10), 48-51, 1981.

(9) S. Fortier, and F. C. Scobey, "Permissible Canal Velocities," Transactions, ASCE, Vol. 89, pp. 940-956, 1926.

(10) H. P. Greenspan and A. V. Johansson, "An Experimental Study of Flow Over an Impounding Dike," Study Application Math 64(3) 211-223, 1981.

(11) H. P. Greenspan and R. E. Young, "Flow Over a Containment Dike," J. Fluid Mechanics, 87:179-192, 1978.

(12) R. H. Haas and H. E. Walker, "Bank Stabilization by Revetments and Dikes," ASCE Proceedings 118, 849-879, 1953.

(13) R. S. Haupt and J. P. Olson, "Case History - Embankment Failure on Soft Varved Silt," ASCE Performance of Earth and Earth-Supported Structures, I-1, 1972.

(14) E. J. Hayter and J. M. Ashish, "Verification of Changes in Flow Regime Due to Dike Breakthrough," J. Waterway, Port, Coastal and Ocean Division, ASCE, 79:729, 1979.

(15) Hydrotechnical Construction (Gidrotekhnicheskoie Stroitelstvo), "The Maximum Permissible Mean Velocity in Open Channels," Moscow, No. 5, PP. 5-7, May 1936.

(16) A. T. Ippen et al., "The Distribution of Boundary Shear Stress in Curved Trapezoidal Channels," Technical Report No. 43, MIT, Department of Civil Engineering, Hydrodynamics Lab, October 1960.

(17) K. Jetter, "Tests on Sand Dikes Protected Against Erosion by Overflowing Water," M.S. Thesis, University of Iowa, Iowa City, 1931.

(18) S. Karaki, K. Mahmood, E. V. Richardson, D. B. Simons, and M. A. Stevens, "River Environment, Hydraulic and Environmental Design Considerations," Federal Highway Administration, CER-73-74EVR-SK-KM-DBS-MAS49, 1974.

(19) V. C. Kartha and H. J. Lentheusser, "Distribution of Tractive Force in Open Channels," Journal of the Hydraulics Division, ASCE, HY7, 1970.

(20) J. W. King and J. B. Beard, "Measuring Rooting of Sodded Turfs," Agronomy Journal, Vol. 61, July-August, 1969, pp. 497-498.

(21) T. W. Lambe and R. V. Whitman, Soil Mechanics, Wiley and Sons, New York, 553 p., 1969.

(22) Y-D Lion, "Hydraulic Erodibility of Two Pure Clay Systems," Ph.D. Dissertation, Department of Civil Engineering, Colorado State University, 1970.

(23) H. K. Liu, J. N. Bradley, and E. J. Plate, "Backwater Effects of Piers and Abutments," Colorado State University, Civil Engineering, CER57HKL10, 364, 1957.

(24) H. S. Manamperi, "Tests of Graded Riprap for Protection of Erodible Material," M.S. Thesis, University of Iowa, Iowa City, 1952.

(25) T. E. Murphy and J. L. Grace, "Riprap for Overflow Embankments," Highway Research Board, HRR-30, 1963.

(26) H. Nasner, "Storm Surge Protection for the Elbe River," in 17th International Conference on Coastal Engineering, Sydney, Australia; Abstracts in Depth, 1980.

(27) E. M. O'Loughlin, S. C. Mehrotra, Y. C. Chang, and J. F. Kennedy, "Scale Effects in Hydraulic Model Tests of Rock Protected Structures," Iowa University, Iowa City Institute of Hydraulic Research, 37 p, 1970.

(28) R. Q. Palmer and J. R. Walker, "Honolulu Reef Runway Dike," in Proceedings of 12th Coastal Engineering Conference, Wash., Vol. 3, Chapter 99, 1629-1646, 1970.

(29) E. Partheniades, "A Study of Erosion and Deposition of Cohesive Soils in Salt Water," Ph.D. Disseration, University of California, Berkeley, 1962.

(30) E. Partheniades, "Erosion and Deposition of Cohesive Soils," Journal of the Hydraulics Division, ASCE, Vol. 91, No. HY1, January 1965, pp. 105-139.

(31) V. C. Patel, "Calibration of the Preston Tube and Limitations on its Use in Pressure Gradients," Journal of Fluid Mechanics, Vol. 23, Part I, 1965.

(32) G. Pilot, "Study of Five Embankment Failures on Soft Soils," ASCE Performance of Earth and Earth-Supported Structures, I-1, 1972.

(33) T. J. Pokrefke and W. Franco, "Investigation of Proposed Dike Systems on the Mississippi River; Rpt. 2, New Madrid Bar Reach; Hydraulic Model Investigation," Vicksburg, MS, USAE Waterways Experiment Station, MPH-70-1, 30 p. 1981.

(34) C. J. Posey, "Flood Erosion Protection for Highway Fills," Trans. ASCE, Paper No. 2871, Vol. 122, pp. 531-555, 1957. Library Reference.

(35) J. H. Preston, "The Determination of Turbulent Skin Friction by Means of Pitot Tubes," Journal of the Royal Aeronautical Society, Vol. 58, 1954.

(36) P. L. Rowlison and G.L. Martin, "Rational Model Describing Slope Erosion," ASCE, Irrigation and Drainage 97, 39-50, 1971.

(37) P. G. Samuels, EMBER - A Numerical Model of an Embanked River," Wallingford Hydraulic Research Station, Report IT 183, 28 p, 1979.

(38) H. W. Schen, S. A. Schumm, and J. D. Nelson, "Methods for Assessment of Stream-Related Hazards to Highways and Bridges," FHWA/RD-80/160, 252 p, 1980.

(39) E. E. Seelye, Data Book for Civil Engineers, Vol. 1, NY, Wiley and Sons, 1966.

(40) P. Singh Shiwendra, "Highway Embankment Doubles As Dam," ASCE, Civil Engineering Journal, 79:80, 1979.

(41) D. B. Simons, Y. H. Chen and A. A. Fiuzat, "Stability Tests of Riprap in Flood Control Channels," for the U.S. Army Waterways Experiment Station, Vicksburg, Mississippi, 1981.

(42) D. B. Simons, G. L. Lewis, and W. G. Field, "Embankment Protection at River Constrictions," Proceedings, Eleventh Annual Bridge Engineering Conference 1970, Department of Civil Engineering, Colorado State University, Fort Collins, Colorado, 1970.

(43) D. B. Simons, G. L. Lewis, "Flood Protection at Bridge Crossings," prepared for Wyoming State Highway Department, Planning and Research Division in Cooperation with the U.S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads, 1970, CER70-71DBS-GLL31.

(44) D. B. Simons, R. M. Li, K. G. Eggert, D. M. Hartley, J.N.H. Ho, and R. Miskimins, "Computer Simulation for Evaluating the Effectiveness of Vegetation Buffer Strips," for Environmental Research Laboratory, Office of Research and Development, U.S. Environmental Protection Agency, 1981.

(45) M. A. Stevens and D. B. Simons, "Stability Analysis for Coarse Granular Material on Slopes," Chapter 17, River Mechanics, edited by H. W. Shen, Colorado State University, November 1971.

(46) E. Tautenhain and S. Kohlhase, "Investigation on Wave Run-Up and Overtopping at Sea Dikes," in Proceedings of International Conference of Water Resources Development, Taipei, Vol. 3, Taipei, Chinese Institute of Engineering, 873-882, 1980.

(47) M. T. Tseng, A. J. Knepp, and R. A. Schmalz, "Evaluation of Flood Risk Factors in the Design of Highway Stream Crossings," Fed. Highway Adm., FHWA-RD-75-54, 1975.

(48) U.S. Army Corps of Engineers, "Earth Embankments," US Government Printing Office, Washington, DC, EM-1110-2-2300, 1959.

(49) U.S. Army Corps of Engineers, "Seepage Control," US Government Printing Office, Washington, DC, EM-1110-2-1091, 1982.

(50) U.S. Bureau of Reclamation, "Method of Computing Erosion Rate for Failure of Earthfill Dams," Unpublished Report, USDI, B.R., Denver, CO, 1965.

(51) F. C. Walker and W. G. Holz, "Control of Embankment Material by Laboratory Testing," ASCE Proceedings 77, No. 108, Dec. 25 p, 1951.

(52) A. Weiss, "Construction Technique of Passing Floods Over Earth Dams," Trans. ASCE, V. 116, p. 1158, 1951.

(53) R. N. Yong and B. P. Warkentine, Introduction to Soil Behavior, MacMillan Co., New York, 1966.

(54) G. K. Young, R. S. Taylor, and L. S. Costello, "Evaluation of the Flood Risk Factor in the Design of Box Culverts," Report No. FHWA-RD-74-11, Vol. 1 of 2, FHWA HRS-42, Washington, DC 20590, September 1970.

☆ U. S. GOVERNMENT PRINTING OFFICE: 1987--181-763--40,175

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH, DEVELOPMENT, AND TECHNOLOGY

The Offices of Research, Development, and Technology (RD&T) of the Federal Highway Administration (FHWA) are responsible for a broad research, development, and technology transfer program. This program is accomplished using numerous methods of funding and management. The efforts include work done in-house by RD&T staff, contracts using administrative funds, and a Federal-aid program conducted by or through State highway or transportation agencies, which include the Highway Planning and Research (HP&R) program, the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board, and the one-half of one percent training program conducted by the National Highway Institute.

The FCP is a carefully selected group of projects, separated into broad categories, formulated to use research, development, and technology transfer resources to obtain solutions to urgent national highway problems.

The diagonal double stripe on the cover of this report represents a highway. It is color-coded to identify the FCP category to which the report's subject pertains. A red stripe indicates category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, and green for category 9.

FCP Category Descriptions

1. Highway Design and Operation for Safety

Safety RD&T addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act. It includes investigation of appropriate design standards, roadside hardware, traffic control devices, and collection or analysis of physical and scientific data for the formulation of improved safety regulations to better protect all motorists, bicycles, and pedestrians.

2. Traffic Control and Management

Traffic RD&T is concerned with increasing the operational efficiency of existing highways by advancing technology and balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, coordinated signal timing, motorist information, and rerouting of traffic.

3. Highway Operations

This category addresses preserving the Nation's highways, natural resources, and community attributes. It includes activities in physical maintenance, traffic services for maintenance zoning, management of human resources and equipment, and identification of highway elements that affect the quality of the human environment. The goals of projects within this category are to maximize operational efficiency and safety to the traveling public while conserving resources and reducing adverse highway and traffic impacts through protections and enhancement of environmental features.

4. Pavement Design, Construction, and Management

Pavement RD&T is concerned with pavement design and rehabilititation methods and procedures, construction technology, recycled highway materials, improved pavement binders, and improved pavement management. The goals will emphasize improvements to highway performance over the network's life cycle, thus extending maintenance-free operation and maximizing benefits. Specific areas of effort will include material characterizations, pavement damage predictions, methods to minimize local pavement defects, quality control specifications, long-term pavement monitoring, and life cycle cost analyses.

5. Structural Design and Hydraulics

Structural RD&T is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highway structures at reasonable costs. This category deals with bridge superstructures, earth structures, foundations, culverts, river mechanics, and hydraulics. In addition, it includes material aspects of structures (metal and concrete) along with their protection from corrosive or degrading environments.

9. RD&T Management and Coordination

Activities in this category include fundamental work for new concepts and system characterization before the investigation reaches a point where it is incorporated within other categories of the FCP. Concepts on the feasibility of new technology for highway safety are included in this category. RD&T reports not within other FCP projects will be published as Category 9 projects.

STORMWATER

Best Management Practices and Detention

for Water Quality, Drainage, and CSO Management

BEN URBONAS
Urban Flood Control District • Denver • Colorado

PETER STAHRE
Malmö Water and Sewer Works • Malmö • Sweden

PTR Prentice Hall, Englewood Cliffs, New Jersey 07632

Library of Congress Cataloging-in-Publication Data

Urbonas, Ben.
Stormwater : best management practices and detention for water quality, drainage, and CSO management / Ben Urbonas, Peter Stahre.
p. cm.
Rev. ed. of: Stormwater detention. 1989, c1990.
Includes bibliographical references and index.
ISBN 0-13-847492-3 :
1. Storm water retention basins. 2. Urban runoff. I. Stahre, Peter. II. Urbonas, Ben. Stormwater detention. III. Title.
TD665.S72 1993
628'.21—dc20 92-17276
CIP

Editorial/production supervision/interior design: *John Morgan*
Cover design: *Ben Santora*
Acquisitions editor: *Michael Hays*
Editorial assistant: *Dana Mercure*
Prepress buyer: *Mary E. McCartney*
Manufacturing buyer: *Susan Brunke*
Proofreader: *Mary Kathryn Bsales*

The publisher offers discounts on this book when ordered in bulk quantities. For more information, write: Special Sales/Professional Marketing. Prentice Hall, Professional & Technical Reference Division, Englewood Cliffs, NJ 07632.

Printed in the United States of America
10 9 8 7 6 5 4 3 2 1

ISBN 0-13-847492-3

Prentice-Hall International (UK) Limited, *London*
Prentice-Hall of Australia Pty. Limited, *Sydney*
Prentice-Hall Canada Inc., *Toronto*
Prentice-Hall Hispanoamericana, S.A., *Mexico*
Prentice-Hall of India Private Limited, *New Delhi*
Prentice-Hall of Japan, Inc., *Tokyo*
Simon & Schuster Asia Pte. Ltd., *Singapore*
Editora Prentice-Hall do Brasil, Ltda., *Rio de Janeiro*

4.3.7 Embankment Loss Analysis

Urban watersheds are typically small, and the amount of total time an embankment is actually overtopped is much shorter than for a large watershed. A detailed embankment loss analysis can quantify the breaching potential of an embankment for a variety of large storm events. A procedure based on the results of highway embankment overtopping studies by Chen and Anderson (1986) for the Federal Highway Administration could be used for this purpose.

Figure 4.14 shows four nomographs that can be used to estimate the soil loss rate of an overtopped bare soil embankment. Two of these provide the erosion rates for a 5-foot (1.5 m) high embankment, while the other two give correction factors for embankments under other design conditions. The procedure for estimating the erosive loss of soil is as follows:

1. Determine the height of the embankment and its soil type (i.e., cohesive or noncohesive).
2. Determine the headwater depth, h, and tailwater, t, above the embankment crest.
3. Compute t/h.
4. Using h and t/h, determine the erosion rate, E_a, from Figure 4.14 for the embankment soil type.
5. Again using Figure 4.14, determine the adjustment factors for embankment height and duration of flow.
6. Compute the erosion rate: $E = K_1 K_2 E_a$.
7. Compute the soil loss for the period the embankment is estimated to be overtopped.

The foregoing procedure may or may not provide an accurate estimate for a particular site. There are many uncertainties in its application, and it is wise to provide safety factors in design. For example, when estimating the embankment loss, you may want to assume that overtopping occurs over only a portion of the crest length.

Here are some suggestions for how to mitigate possible adverse impacts of detention pond dam overtopping whenever the emergency spillway is of less capacity than the probable maximum flood:

- Construct the embankment using erosion resistant material. For example, concrete rubble or large rock embedded in compacted clay soils on the downstream side of the dam may be able to withstand considerable overtopping for brief periods of time without sudden and catastrophic failure.

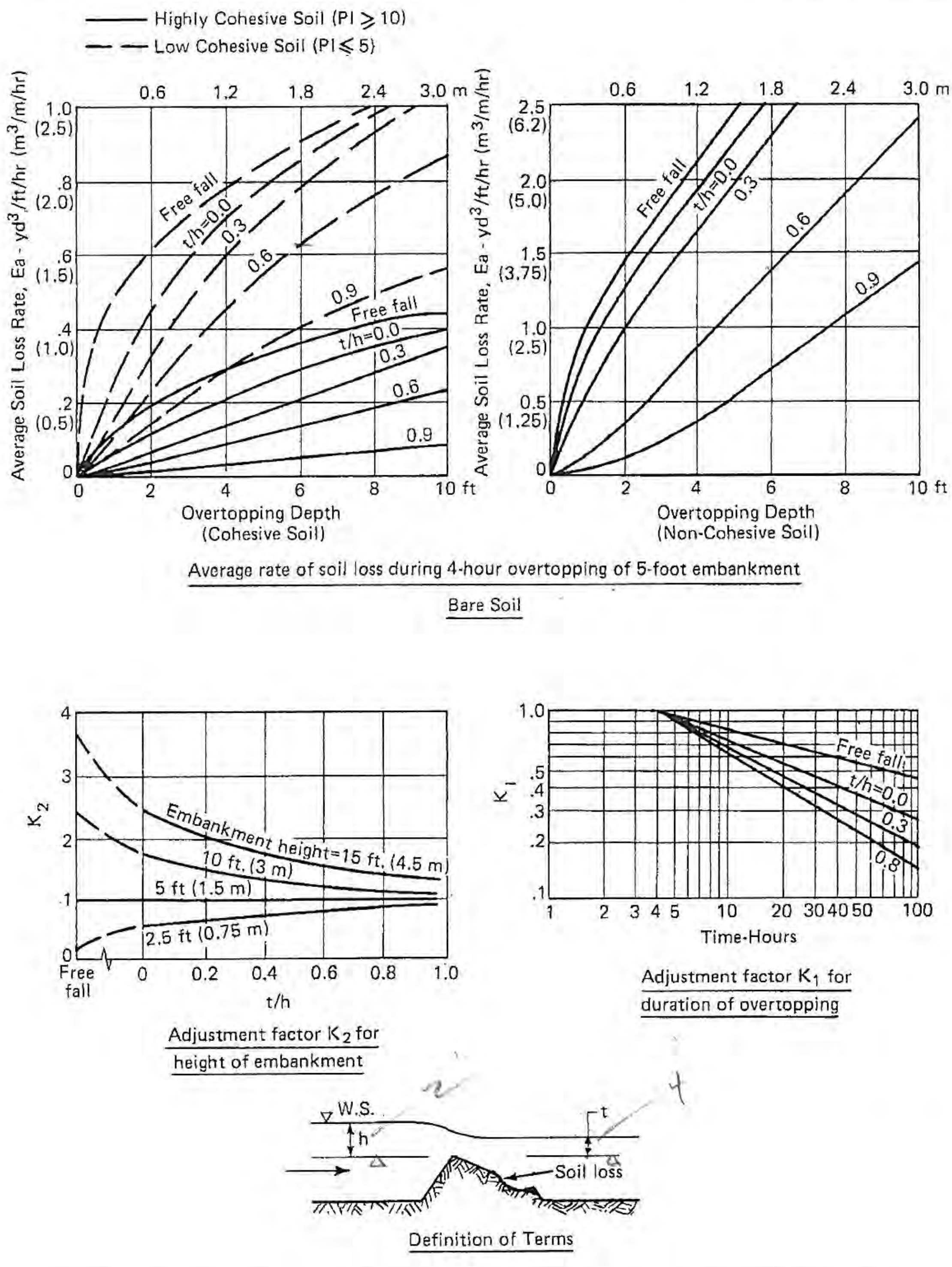


Figure 4.14 Soil erosion loss nomographs for overtopped bare soil highway embankments. *(After Chen and Anderson, 1986.)*

- Use noneroding materials on the downstream portion of the embankment such as soil cement, rollcrete, interlocking concrete elements or concrete paving.
- Design the embankment with an extra wide crest or construct a paved road on top.
- Flatten the downstream face of the dam and revegetate with native turf-forming grasses.
- Enlarge the conveyance capacity downstream of the dam.
- Excavate the pond to eliminate or reduce the height of the embankment.

4.4 OPERATION AND MAINTENANCE

If you do not plan to maintain it, do not build it. That should be the golden rule in public works. As a result, all detention ponds should have maintainability built into them. This includes good maintenance access to all parts of the ponding area for mechanized maintenance equipment. Although this suggestion may seem trivial, one of the most frequently observed deficiencies is inadequate maintenance access. Can it be built, and can it be maintained? These two questions should always be asked as the detention facilities are being designed. If the answer is no to either question, modify the design.

In this section we discuss some of the operation and maintenance considerations for detention ponds. The following topics are discussed:

- general inspection and maintenance,
- algae and aquatic plants,
- sediments,
- floating debris and pollutants,
- general housekeeping, and
- erosion.

4.4.1 General Inspection and Maintenance

The responsibility for maintenance rests with the owner of the detention pond. As part of the design, provide an operation and maintenance manual to the owner. It is a good idea for the owner to have one prepared for already existing facilities. Such a manual does not need to be complicated. It merely needs to serve as a checklist of things to look for, and to do, on a regular basis. A more detailed, but more valuable, manual could contain copies of as-built drawings for key elements of the pond, a list of replacement parts, emergency procedures in case of very large floods, names and telephone numbers of key individuals, etc. Obviously, the level of detail will vary with the size and the complexity of the installation.

debris and trash removal and regular mowing of the grass around the pond and within the detention basin.

The frequency of such maintenance will be governed by the specific uses of detention facilities and whether they are located close to residential or commercial areas. One cannot offer standardized guidelines for scheduling such activities, since they vary between installations and between various communities. What is important, however, is to anticipate and to budget for routine maintenance and housekeeping when new detention facilities are being installed.

REFERENCES

ASCE, *Stormwater Detention Outlet Control Structures,* Final Report of the Task Committee on the Design of Outlet Structures, American Society of Civil Engineers, New York, 1985.

BRULO, A. T., KIBLER, D. F., AND MILLER, A. C., "Evaluation of Two Stage Outlet Hydraulics," *Proceedings of the ASCE Hydraulics Division Conference on Water for Resources Development,* New York, 1984.

CHEN, Y.-H., AND ANDERSON, B. A., *Analysis of Data in Literature for Estimating Embankment Damage Due to Flood Overtopping,* Simons, Li & Associates, Inc. Report to U.S. Department of Transportation, Federal Highway Administration, DC-FHA-01, Washington, D.C., 1986.

EDWARDS, K. L., "Acceptance and/or Resistance to Detention Ponds," *Proceedings of the Conference on Stormwater Detention Facilities,* American Society of Civil Engineers, New York, 1982.

EPA, *Results of the Nationwide Urban Runoff Program,* NTIS Access Number: PB84-185552, Environmental Protection Agency, Washington, D.C., 1983.

KAMELDUSKI, D. E., AND MCCUEN, R. H., "Evaluation of Alternative Stormwater Detention Policies," *Journal of Water Resources Planning and Management Division,* Vol. 105, pp 171–86, American Society of Civil Engineers, New York, Sept. 1979.

KROPP, R. H., "Water Quality Enhancement Design Techniques," *Proceedings of the Conference on Stormwater Detention Facilities,* American Society of Civil Engineers, New York, 1982.

POERTNER, H. G, *Practices in Detention of Urban Stormwater Runoff,* American Public Works Association, Special Report Number 43, 1974.

SCHROEDER, G., *Agricultural Water Engineering,* 3rd ed., Springer-Verlag, Berlin, 1958. (In German).

STUBCHEAR, JAMES M., "Stormwater Basins in Santa Barbara County," *Proceedings of the Conference on Stormwater Detention Facilities,* American Society of Civil Engineers, New York, 1982.

UDFCD, *Urban Storm Drainage Criteria Manual,* Vol. 3 (draft), Urban Drainage and Flood Control District, Denver, 1992.

URBONAS, B. R., AND GLIDDEN, M. W., "Potential Effectiveness of Detention Poli-

Attachment B
Dam Breach Analysis per Colorado State Engineer’s Office (SEO) Guidance

ESTIMATION OF DAM BREACH PARAMETERS
USING THE MACDONALD & LANGRIDGE-MONOPOLIS OR WASHINGTON STATE METHODS
WITH ALL FAILURE TIMES ESTIMATED BY WASHINGTON STATE METHOD

PROJECT: 121-030.000 Lateral Berm with 6-feet overtopping

BREACH INPUT PARAMETERS:

Select Embankment Type From Drop-Down Menu: EARTHEN (NON-COHESIVE)

Height of water over base elevation of breach (H_w) =	26.0	Feet
Volume of water stored in reservoir at time of failure (V_w) =	1100.0	Acre-Feet
Reservoir Surface Area at H_w (S_a) =	27.5	Acres
Crest width of dam (C) =	300.0	Feet
Height of breach from dam crest to base elevation of breach (H_b) =	19.7	Feet
Slope of upstream dam face (Z_u) =	2.0	Z(H):1(V)
Slope of downstream dam face (Z_d) =	3.0	Z(H):1(V)
Breach side-slope ratio (Z_b) =	2.0	Z(H):1(V)
Piping Orifice Coefficient (C_p) =	0.64	Used To Calculate Peak Discharge Through Piping Hole
Dam Size Class:	Small	Assumes Full Reservoir At Time of Breach

CALCULATED BREACH CHARACTERISTICS:

Breach Formation Factor (BFF) =	28600	
Embankment Volume Eroded (V_{er}) =	10125.9	Cubic Yards
Average Dam Width (W_{avg}) =	349.3	Feet (In Direction of Flow)
Average Breach Width (B_{avg}) =	39.7	Feet
Bottom Width of Breach (B_b) =	0.3	Feet
Breach Formation Time (T_f) =	0.55	Hours
Storage Intensity (SI) =	42.3	Acre Feet/Foot
Peak Breach Discharge (Q_p) =	10087	Cubic Feet per Second

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (Bavg/Hb) =	2.02	If (Bavg/Hb) > 0.6, Full Breach Development is Anticipated
Erosion Rate (ER), Calculated as (Bavg/Tf) =	71.8	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/Hw) =	2.8	If 1.6 < (ER/Hw) < 21, Erosion Rate is Assumed Reasonable

ESTIMATION OF DAM BREACH PARAMETERS
USING THE FROEHLICH 2008 METHOD

PROJECT: 121-030.000 Lateral Berm with 6-feet overtopping

BREACH INPUT PARAMETERS:

Select Failure Mode From Drop-Down Menu: OVERTOPPING

Height of water over base elevation of breach (H_w) =	36.0	Feet
Volume of water in the reservoir at the time of failure (V_w) =	1,100.0	Acre-Feet
Reservoir Surface Area at Hw (A_s) =	27.5	Acres
Height of breach (H_b) =	30.0	Feet
Failure Mode Factor (K_o) =	1.3	
Breach Side-Slope Ratio (Z_b) =	1	Z(H):1(V)
Dam Size Class:	Small	Assumes Full Reservoir At Time of Breach.

CALCULATED BREACH CHARACTERISTICS:

Average Breach Width (B_{avg}) =	115.4	Feet
Bottom Width of Breach (B_b) =	85.4	Feet
Breach Formation Time (T_f) =	0.71	Hours
Storage Intensity (SI) =	30.6	Acre Feet/Foot
Predicted Peak Flow (Q_p) =	13981	Cubic Feet per Second

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (B_{avg}/H_b) =	3.85	If (B_{avg}/H_b) > 0.6, Full Breach Devlopment is Anticipated
Erosion Rate (ER), Calculated as (B_{avg}/T_f) =	161.6	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/H_w) =	4.5	If 1.6 < (ER/H_w) < 21, Erosion Rate is Assumed Reasonable

ESTIMATION OF DAM BREACH PARAMETERS
USING THE FROEHLICH 2008 METHOD

PROJECT: 121-030.000 Lateral Berm with 6-feet overtopping

BREACH INPUT PARAMETERS:

Select Failure Mode From Drop-Down Menu:	PIPING	
Height of water over base elevation of breach (H_w) =	36.0	Feet
Volume of water in the reservoir at the time of failure (V_w) =	1,100.0	Acre-Feet
Reservoir Surface Area at Hw (A_s) =	27.5	Acres
Height of breach (H_b) =	30.0	Feet
Failure Mode Factor (K_o) =	1	
Breach Side-Slope Ratio (Z_b) =	0.7	Z(H):1(V)
Dam Size Class:	Small	Assumes Full Reservoir At Time of Breach.

CALCULATED BREACH CHARACTERISTICS:

Average Breach Width (B_{avg}) =	88.8	Feet
Bottom Width of Breach (B_b) =	67.8	Feet
Breach Formation Time (T_f) =	0.71	Hours
Storage Intensity (SI) =	30.6	Acre Feet/Foot
Predicted Peak Flow (Q_p) =	14764	Cubic Feet per Second

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (B_{avg}/H_b) =	2.96	If (B_{avg}/H_b) > 0.6, Full Breach Devlopment is Anticipated
Erosion Rate (ER), Calculated as (B_{avg}/T_f) =	124.3	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/H_w) =	3.5	If 1.6 < (ER/H_w) < 21, Erosion Rate is Assumed Reasonable

ESTIMATION OF DAM BREACH PARAMETERS
USING THE MACDONALD & LANGRIDGE-MONOPOLIS OR WASHINGTON STATE METHODS
WITH ALL FAILURE TIMES ESTIMATED BY WASHINGTON STATE METHOD

PROJECT: 121-030.000 Lateral Berm with 6-feet overtopping

BREACH INPUT PARAMETERS:

Select Embankment Type From Drop-Down Menu: EARTHEN (NON-COHESIVE)

Height of water over base elevation of breach (H_w) =	21.0	Feet
Volume of water stored in reservoir at time of failure (V_w) =	1100.0	Acre-Feet
Reservoir Surface Area at H_w (S_a) =	27.5	Acres
Crest width of dam (C) =	300.0	Feet
Height of breach from dam crest to base elevation of breach (H_b) =	15.0	Feet
Slope of upstream dam face (Z_u) =	2.0	Z(H):1(V)
Slope of downstream dam face (Z_d) =	3.0	Z(H):1(V)
Breach side-slope ratio (Z_b) =	2.0	Z(H):1(V)
Piping Orifice Coefficient (C_p) =	0.64	Used To Calculate Peak Discharge Through Piping Hole
Dam Size Class:	Small	Assumes Full Reservoir At Time of Breach

CALCULATED BREACH CHARACTERISTICS:

Breach Formation Factor (BFF) =	23100	
Embankment Volume Eroded (V_{er}) =	8590.4	Cubic Yards
Average Dam Width (W_{avg}) =	337.5	Feet (In Direction of Flow)
Average Breach Width (B_{avg}) =	45.8	Feet
Bottom Width of Breach (B_b) =	15.8	Feet
Breach Formation Time (T_f) =	0.52	Hours
Storage Intensity (SI) =	52.4	Acre Feet/Foot
Peak Breach Discharge (Q_p) =	8530	Cubic Feet per Second

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (Bavg/Hb) =	3.05	If (Bavg/Hb) > 0.6, Full Breach Development is Anticipated
Erosion Rate (ER), Calculated as (Bavg/Tf) =	87.8	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/Hw) =	4.2	If 1.6 < (ER/Hw) < 21, Erosion Rate is Assumed Reasonable

ESTIMATION OF DAM BREACH PARAMETERS
USING THE FROEHLICH 2008 METHOD

PROJECT: 121-030.000 Lateral Berm with 6-feet overtopping

BREACH INPUT PARAMETERS:

Select Failure Mode From Drop-Down Menu: OVERTOPPING

Height of water over base elevation of breach (H_w) =	21.0	Feet
Volume of water in the reservoir at the time of failure (V_w) =	1,100.0	Acre-Feet
Reservoir Surface Area at Hw (A_s) =	27.5	Acres
Height of breach (H_b) =	15.0	Feet
Failure Mode Factor (K_o) =	1.3	
Breach Side-Slope Ratio (Z_b) =	1	Z(H):1(V)
Dam Size Class:	Small	Assumes Full Reservoir At Time of Breach.

CALCULATED BREACH CHARACTERISTICS:

Average Breach Width (B_{avg}) =	112.2	Feet
Bottom Width of Breach (B_b) =	97.2	Feet
Breach Formation Time (T_f) =	1.43	Hours
Storage Intensity (SI) =	52.4	Acre Feet/Foot
Predicted Peak Flow (Q_p) =	3411	Cubic Feet per Second

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (B_{avg}/H_b) =	7.48	If (B_{avg}/H_b) > 0.6, Full Breach Devlopment is Anticipated
Erosion Rate (ER), Calculated as (B_{avg}/T_f) =	78.6	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/H_w) =	3.7	If 1.6 < (ER/H_w) < 21, Erosion Rate is Assumed Reasonable

6

ESTIMATION OF DAM BREACH PARAMETERS
USING THE FROEHLICH 2008 METHOD

PROJECT: 121-030.000 Lateral Berm with 6-feet overtopping

BREACH INPUT PARAMETERS:

Select Failure Mode From Drop-Down Menu:	PIPING	
Height of water over base elevation of breach (H_w) =	21.0	Feet
Volume of water in the reservoir at the time of failure (V_w) =	1,100.0	Acre-Feet
Reservoir Surface Area at Hw (A_s) =	27.5	Acres
Height of breach (H_b) =	15.0	Feet
Failure Mode Factor (K_o) =	1	
Breach Side-Slope Ratio (Z_b) =	0.7	Z(H):1(V)
Dam Size Class:	Small	Assumes Full Reservoir At Time of Breach.

CALCULATED BREACH CHARACTERISTICS:

Average Breach Width (B_{avg}) =	86.3	Feet
Bottom Width of Breach (B_b) =	75.8	Feet
Breach Formation Time (T_f) =	1.43	Hours
Storage Intensity (SI) =	52.4	Acre Feet/Foot
Predicted Peak Flow (Q_p) =	3890	Cubic Feet per Second

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (B_{avg}/H_b) =	5.76	If (B_{avg}/H_b) > 0.6, Full Breach Devlopment is Anticipated
Erosion Rate (ER), Calculated as (B_{avg}/T_f) =	60.5	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/H_w) =	2.9	If 1.6 < (ER/H_w) < 21, Erosion Rate is Assumed Reasonable

ESTIMATION OF DAM BREACH PARAMETERS
USING THE MACDONALD & LANGRIDGE-MONOPOLIS OR WASHINGTON STATE METHODS
WITH ALL FAILURE TIMES ESTIMATED BY WASHINGTON STATE METHOD

PROJECT: 121-030.000 Lateral Berm with 2-feet overtopping

BREACH INPUT PARAMETERS:

Select Embankment Type From Drop-Down Menu: EARTHEN (NON-COHESIVE)

Height of water over base elevation of breach (H_w) =	20.0	Feet
Volume of water stored in reservoir at time of failure (V_w) =	1100.0	Acre-Feet
Reservoir Surface Area at H_w (S_a) =	27.5	Acres
Crest width of dam (C) =	300.0	Feet
Height of breach from dam crest to base elevation of breach (H_b) =	18.0	Feet
Slope of upstream dam face (Z_u) =	2.0	Z(H):1(V)
Slope of downstream dam face (Z_d) =	3.0	Z(H):1(V)
Breach side-slope ratio (Z_b) =	2.0	Z(H):1(V)
Piping Orifice Coefficient (C_p) =	0.64	Used To Calculate Peak Discharge Through Piping Hole
Dam Size Class:	Small	Assumes Full Reservoir At Time of Breach

CALCULATED BREACH CHARACTERISTICS:

Breach Formation Factor (BFF) =	22000	
Embankment Volume Eroded (V_{er}) =	8273.6	Cubic Yards
Average Dam Width (W_{avg}) =	344.9	Feet (In Direction of Flow)
Average Breach Width (B_{avg}) =	36.1	Feet
Bottom Width of Breach (B_b) =	0.2	Feet
Breach Formation Time (T_f) =	0.51	Hours
Storage Intensity (SI) =	55.0	Acre Feet/Foot
Peak Breach Discharge (Q_p) =	6952	Cubic Feet per Second

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (Bavg/Hb) =	2.01	If (Bavg/Hb) > 0.6, Full Breach Development is Anticipated
Erosion Rate (ER), Calculated as (Bavg/Tf) =	70.1	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/Hw) =	3.5	If 1.6 < (ER/Hw) < 21, Erosion Rate is Assumed Reasonable

7

ESTIMATION OF DAM BREACH PARAMETERS
USING THE FROEHLICH 2008 METHOD

PROJECT: 121-030.000 Lateral Berm with 2-feet overtopping

BREACH INPUT PARAMETERS:

Select Failure Mode From Drop-Down Menu: OVERTOPPING

Height of water over base elevation of breach (H_w) =	32.0	Feet
Volume of water in the reservoir at the time of failure (V_w) =	1,100.0	Acre-Feet
Reservoir Surface Area at Hw (A_s) =	27.5	Acres
Height of breach (H_b) =	30.0	Feet
Failure Mode Factor (K_o) =	1.3	
Breach Side-Slope Ratio (Z_b) =	1	Z(H):1(V)
Dam Size Class:	Small	Assumes Full Reservoir At Time of Breach.

CALCULATED BREACH CHARACTERISTICS:

Average Breach Width (B_{avg}) =	115.4	Feet
Bottom Width of Breach (B_b) =	85.4	Feet
Breach Formation Time (T_f) =	0.71	Hours
Storage Intensity (SI) =	34.4	Acre Feet/Foot
Predicted Peak Flow (Q_p) =	12635	Cubic Feet per Second

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (B_{avg}/H_b) =	3.85	If (B_{avg}/H_b) > 0.6, Full Breach Devlopment is Anticipated
Erosion Rate (ER), Calculated as (B_{avg}/T_f) =	161.6	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/H_w) =	5.1	If 1.6 < (ER/H_w) < 21, Erosion Rate is Assumed Reasonable

ESTIMATION OF DAM BREACH PARAMETERS
USING THE FROEHLICH 2008 METHOD

PROJECT: 121-030.000 Lateral Berm with 2-feet overtopping

BREACH INPUT PARAMETERS:

Select Failure Mode From Drop-Down Menu:	PIPING	
Height of water over base elevation of breach (H_w) =	32.0	Feet
Volume of water in the reservoir at the time of failure (V_w) =	1,100.0	Acre-Feet
Reservoir Surface Area at Hw (A_s) =	27.5	Acres
Height of breach (H_b) =	30.0	Feet
Failure Mode Factor (K_o) =	1	
Breach Side-Slope Ratio (Z_b) =	0.7	Z(H):1(V)
Dam Size Class:	Small	Assumes Full Reservoir At Time of Breach.

CALCULATED BREACH CHARACTERISTICS:

Average Breach Width (B_{avg}) =	88.8	Feet
Bottom Width of Breach (B_b) =	67.8	Feet
Breach Formation Time (T_f) =	0.71	Hours
Storage Intensity (SI) =	34.4	Acre Feet/Foot
Predicted Peak Flow (Q_p) =	13196	Cubic Feet per Second

RESULTS CHECK:

Average Breach Width Divided by Height of Breach (B_{avg}/H_b) =	2.96	If (B_{avg}/H_b) > 0.6, Full Breach Devlopment is Anticipated
Erosion Rate (ER), Calculated as (B_{avg}/T_f) =	124.3	
Erosion Rate Divided by Height of Water Over Base of Breach (ER/H_w) =	3.9	If 1.6 < (ER/H_w) < 21, Erosion Rate is Assumed Reasonable

STATE OF COLORADO
DEPARTMENT OF NATURAL RESOURCES
DIVISION OF WATER RESOURCES

OFFICE OF THE STATE ENGINEER
DAM SAFETY BRANCH

GUIDELINES FOR DAM BREACH ANALYSIS

February 10, 2010

Telephone (303) 866-3581
Facsimile (303) 866-3589

1313 Sherman Street
Room 818 Centennial Building
Denver, Colorado

Website:
http://water.state.co.us

The Simple approach is considered moderately conservative. In most cases, it is not as conservative as the Screening level because the breach hydrograph typically has a smaller peak due to the parametric modeling of the breach formation, and the hydrologic routing typically results in flood wave attenuation by the time it reaches critical locations. A steady-state hydraulic model can then be used to accurately predict hydraulic conditions at critical locations. The results of the steady-state hydraulic model can be used to create inundation mapping for Emergency Action Plans. If this method results in a borderline situation, it may be necessary to employ a more advanced approach.

6.3 Intermediate

The Intermediate approach lies between the simple approach and advanced approach in accuracy and sophistication. Similar to the simple approach, it uses empirical equations to determine the breach parameters (geometry and failure time). Those dimensions are then input into a hydrologic parametric model (HEC-HMS or HEC-1) to calculate the breach flood hydrograph which is then input into a hydraulic model (HEC-RAS) in an unsteady flow simulation to route the flood downstream and calculate the hydraulic conditions at critical locations.

This approach may not be as accurate as the advanced approach for piping failures of smaller dams because the usage of HEC-1 and HEC-HMS to develop the dam break hydrographs may not model this process as accurately as HEC-RAS or DAMBRK. However, it may be just as accurate as the advanced approach for overtopping scenarios or for piping failures of larger dams. This approach is a viable option for developing flood inundation mapping for Emergency Action Plans.

6.4 Advanced

The Advanced approach is the most rigorous level of analysis. Similar to the Simple approach, it uses empirical equations to determine the breach parameters (geometry and failure time). Those dimensions are then input into a hydraulic parametric model (HEC-RAS or DAMBRK) to calculate the breach flood For DAMBRK the hydrograph is then input into (HEC-RAS) in an unsteady flow simulation to route the flood downstream and calculate the hydraulic conditions at critical locations. For HEC-RAS, the dam failure simulation and downstream routing is performed in the same simulation.

The increased accuracy of the Advanced approach comes at the expense of more time required to develop, debug and refine the unsteady hydraulic model. This level of analysis can be time consuming, particularly if the downstream drainage is complex and critical sections are located well downstream.

7.0 Recommendations for Dam Breach Analysis

The recommendations presented herein for modeling dam breaches are intended to provide the most realistic dam breach flood estimates while still being appropriately conservative. For the purposes of these recommendations, the term "conservative" means an analysis that tends to overestimate the magnitude and impacts of the dam breach flood. For example, an increase in the estimate of average breach width for a given development time leads to an increase in the peak breach discharge and associated impacts downstream. Being appropriately conservative at this time is warranted because of the need for better physically-based modeling of the erosion processes of dam failures, which is still in the developmental stage. These recommendations are based on case studies performed on a range of dams within Colorado. A summary of the case study results is presented in Appendix A.

7.1 Breach Parameter Estimation

7.1.1 Empirical Methods

The MacDonald & Langridge-Monopolis (1984), Washington State (2007) and Froehlich (2008) methods are the recommended empirical tools for predicting dam breach parameters within the State of Colorado. The appropriate equations with English units are summarized in Table 2.

The MacDonald & Langridge-Monopolis method computes a volume of embankment eroded during breach formation, based on the product of the reservoir volume (V_w) and maximum water depth (H_w). This product, termed the Breach Formation Factor (BFF), loosely represents the erosive potential of the water stored in the reservoir. The breach dimensions are calculated based on the volume of embankment material eroded and the dam geometry. This method considers the dam geometry (height, crest width, and embankment slopes), and the breach development time computed is directly related to the embankment volume eroded. Wahl (1998) provides an equation for this relationship, using an envelope curve of the data, thereby making breach development time estimates conservative. This method also distinguishes between earth-fill dams and rockfill dams.

Washington State (2007) took the MacDonald & Langridge-Monopolis method and adjusted it based upon whether the dam is made of cohesionless or cohesive material. Comparing the predicted earth-fill embankment volume eroded, the Washington State cohesionless equation results in a slightly larger eroded volume estimate than the best fit curve estimates of the MacDonald & Langridge-Monopolis method. As would be expected, results from the Washington State cohesive soil equation show less embankment volume eroded than the MacDonald & Langridge-Monopolis method. The Washington State method estimates the breach development time for cohesionless soil using the MacDonald & Langridge-Monopolis method and developed its own equation to estimate breach development time for cohesive soil using a best fit to the midpoint of the data instead of an envelope equation. Discussion in the Washington State Technical Note suggests a minimum breach formation factor (BFF), of 100 ac-ft^2. This method therefore appears more suited to Small or Large dams, while the MacDonald & Langridge-Monopolis method appears to be more appropriate for Minor dams and some small dams with a BFF less than 100 ac-ft^2.

The Froehlich (2008) method is dependent only on the volume of the reservoir, height of the breach and the assumed breach side-slope. The method distinguishes between piping and overtopping failures using a variable coefficient termed the Failure Mode Factor, K_o. Everything else being equal, an overtopping analysis produces a larger breach section compared to a piping failure analysis. The Froehlich method breach development time does not distinguish between overtopping or piping breach failure modes. The development time estimate is inversely related to the breach height while being directly related to the reservoir volume. This means dams with greater height tend to produce shorter failure times for a given reservoir volume which appears to be a valid conclusion considering the greater head driving the breach formation.

Table 2 – Summary of Recommended Empirical Equations (English Units)

Breach Parameters	MacDonald & Langridge-Monopolis (1984)	Washington (2007)	Froehlich (2008)
Volume Eroded V_{er} (yd^3)	$V_{er} = 3.264BFF^{0.77}$ (best fit all data)	$V_{er} = 3.75BFF^{0.77}$ (cohesionless dams)	
	$V_{er} = 0.714BFF^{0.852}$ (rockfill)	$V_{er} = 2.5BFF^{0.77}$ (cohesive dams)	
Average Breach Width B_{avg} (ft)	$B_{avg} = \frac{V_{er}}{(H_b \times W_{avg})}$		$B_{avg} = 8.239K_o V_w^{0.32} H_b^{0.04}$ K_o=1.0 for piping K_o=1.3 for overtopping
Breach Side slopes Z_b (H:V)	2.0:1		0.7:1 - piping 1.0:1 - overtopping
Breach Development Time T_f (hr)	$T_f = 0.016V_{er}^{0.364}$	$T_f = 0.02V_{er}^{0.36}$ (cohesionless)	$T_f = 3.664\sqrt{\frac{V_w}{gH_b^2}}$
		$T_f = 0.036V_{er}^{0.36}$ (cohesive)	

Suggested Methods to Validate the Parameters Calculated using Empirical Methods:

On a case by case basis, judgment is needed with the predicted parameters calculated using the recommended methods presented here. There are a few general tools used to validate the predicted parameters:

1. An estimate of linear erosion rate can be used to check the validity of the failure time. Linear erosion rate (ER) is defined as the B_{avg}/T_f. Von Thun and Gillette (1990) suggests the minimum allowable erosion rate related to the height of the water above the breach bottom, can be empirically defined as $4H_w$ and the maximum erosion rate related to the water depth is $200 + 4H_w$. However, the data set used to develop the empirical parameters suggest a minimum ER of $1.6H_w$. **If the T_f, B_{avg}, and H_w computed by the empirical methods listed above produces an ER/H_w much less than 1.6, then either the T_f is too long or B_{avg} is too small** and adjustments are needed or a different method selected. Likewise, the maximum ER/H_w in the data set was only 21, which is considerably less than upper limit defined by Von Thun and Gillette (1990) (greater than 200). The average ER/H_w computed from the database was 6.7. **Therefore, if the ER/H_w ratio is greater than 21, then the parameters are considered suspect.**

2. Von Thun and Gillette (1990) suggests that B_{avg}/H_w cannot be less than 2.5. However, the data set, especially for piping, shows B_{avg}/H_w less than 2.5 in many instances. In fact, it is near 1.0 in several cases and less than 1.0 in a few instances. The minimum B_{avg}/H_w for the data set was 0.6 and the minimum B_{avg}/H_b was 0.5. This ratio is highly dependent on storage-intensity ($SI = V_w/H_w$) and with a relatively small reservoir volume relative to the dam height (low storage intensity), the reservoir evacuates quickly and does not allow for the breach to widen. Piping failure of a dam with a very low storage-intensity may evacuate the reservoir through the piping hole without a full rectangular or trapezoidal breach forming. Paquir, et.al, (post 1995) suggested that the piping hole width has to reach 2/3 of the dam height above the bottom of the pipe before the roof of the piping hole collapses

and the breach transitions into a full breach formation. In any event, it is very possible to have a B_{avg} less than H_b. **If the ratio of B_{avg}/H_b is less than 0.6, then the method is suspect or the reservoir is so small that only a piping hole will form.** (See Section 7.1.1.1 for additional discussion on this topic)

3. For dams where two empirical methods may be appropriate, the average flow velocity through the breach may be used to validate one method over another. This technique requires that the average velocity be determined when the breach has reached its final width. The velocity is calculated based on the reservoir level and flow at that time. In theory, since the breach has reached its maximum size, there should not be enough head in the reservoir and/or flow through the breach to cause significantly more erosion to occur. When comparing two methods, the method that yields the smallest velocity at the final breach configuration is probably the most appropriate method. It is necessary to run a parametric model to use this validation technique, but it should help determine which method is most appropriate. The reservoir level, flow and velocity is easily determined with the HEC-RAS model, but can also be estimated from the other models if the reservoir level and the flow through the breach is known.

4. In some cases, the valley geometry is narrow and will control the breach width. **One cannot have a B_{avg} wider than the valley at that particular elevation.** Also, the breach bottom width and side slopes must be selected to ensure the bottom width of the breach is not wider than the valley width at that elevation.

5. The Federal Energy Regulatory Commission's Dambreak Studies guideline (FERC, 1993) provides a range of reasonable values for breach width, side slope and failure time for earthen embankments. Breach Width typically falls between one to five times the dam height; failure time typically falls within 0.1 - 1.0 hours; and breach side slopes typically range between 1H:4V to 1H:1V. The same guidelines also have typical breach parameters for other types of dams including arch, buttress, masonry, gravity, monoliths, timber crib and slag/refuse dams.

The Froehlich (2008) method is recommended for Small or Large dams with a volume greater than 100 AF, as it yields conservative, but reasonable results. The MacDonald & Langridge-Monopolis (1984) and/or Washington State (2007) methods may also be appropriate for certain situations on a case-by-case basis. The value of storage intensity provides a check for which method is most appropriate (see Table 3 below).

For Minor dams and Small dams with a capacity of less than 100 AF, use of the MacDonald & Langridge-Monopolis (1984) method is recommended to estimate volume eroded and breach geometry. Breach development time estimates for Minor and Small dams should be computed by the Washington State method.

The MacDonald & Langridge-Monopolis (1984) and Washington State (2007) methods tend to yield a very narrow breach width for Small and Minor dams that have breached due to overtopping. The Froehlich (2008) method appears to give more reliable results and is recommended for all sizes of dams to calculate the breach parameters for an overtopping failure mode.

If the dam material is known or can be assumed to be cohesionless, then the Washington State (2007) method can be used outright to calculate V_{er} and T_f, but the reasonableness of the results need to be checked. For instance if a small or minor sized dam built with cohesionless soils, produces a B_{avg}/H_b value less than 0.6 with the Washington State (2007) method, the Froehlich (2008) method may be more useful. Likewise, the MacDonald and Langridge-Monopolis (1984) method has a specific equation to estimate V_{er} for rockfill dams.

Erosion rate (ER) guidelines of 1 < $ER/H_w < 21$, where $ER=B_{avg}/T_f$, can be used as check of the methods and the parameters adjusted accordingly. Table 3 summarizes the generally appropriate empirical methods for varying dam sizes and storage intensities. This is only a guide and engineering judgment is needed on a case-by-case basis considering the ER/H_w and B_{avg}/H_b guidelines mentioned above.

Table 3 - Guide of Appropriate Empirical Methods for Various Dam Sizes and Storage-Intensities

Dam Size	Storage Intensity $(SI) = V_w/H_w$		
	Low ($SI < 5$)	**Medium** ($5 < SI < 20$)	**High** ($SI > 20$)
Minor	*MacDonald & Langridge-Monopolis with Washington State failure time. Froehlich for Overtopping.	*MacDonald & Langridge-Monopolis with Washington State failure time. Froehlich for Overtopping.	*MacDonald & Langridge-Monopolis with Washington State failure time. Froehlich for Overtopping.
Small	*MacDonald & Langridge-Monopolis with Washington State failure time and possibly Froehlich (case-by-case). Froehlich for Overtopping.	Froehlich and possibly *MacDonald & Langridge-Monopolis with Washington State failure time (case-by-case).	Froehlich for geometry and failure time.
Large	Froehlich. The side slopes may need to be adjusted to yield a reasonable bottom width.	Froehlich and possibly *MacDonald & Langridge-Monopolis with Washington State failure time (case-by-case).	Froehlich and possibly *MacDonald & Langridge-Monopolis with Washington State failure time (case-by-case).
Comments	Parameters likely need to be adjusted with judgment on a case-by-case basis – may need to be modeled as piping hole for Small and Minor dams.	Both Froehlich and *MacDonald & Langridge-Monopolis seem to work for Small and Large dams in the middle range of SI. Engineering judgment is needed on a case-by-case basis.	It is important to look at valley and dam constraints as the computed parameters may exceed the valley width and/or dam length.
References	Froehlich (2008) MacDonald & Langridge-Monopolis (1984) Washington State (2007)		
* Where the MacDonald & Langridge-Monopolis Method is referenced as a recommendation, this only applies for embankments constructed of cohesive materials. The Washington State Method is preferred for cohesionless earthen embankments.			

7.1.1.1 Piping Failure Considerations with Empirical Methods

For Small and Minor dams with low storage intensities (SI less than 5) that are built with cohesive soils, it is possible that a piping failure could occur and drain the reservoir without fully breaching the dam (i.e. collapsing the crest). This situation is evident when the MacDonald & Langridge-Monopolis and Washington State empirical method for establishing the breach parameters shows that the volume eroded (V_{er}) results in a corresponding B_{avg}/H_b of less than about 0.5. This phenomenon is common for Small dams with a volume less than 100 AF and SI less than about 2.5, and Minor dams when SI is less than about 1.5. When this occurs, it is possible to calculate the maximum piping-hole size (assumed to be square) from the volume of embankment eroded. This piping-only failure mode does not apply to dams built with cohesionless soils. If the MacDonald & Langridge-Monopolis and Washington State empirical method produces a B_{avg}/H_b value of less than 0.6 for a dam built with cohesionless soils a full breach should be assumed and another empirical method such as the Froehlich 2008 should be used.

When the computed maximum pipe dimension, D, is less than $0.6H_b$ then this mode of failure can be modeled using a sluice gate and a square opening with HEC-RAS. The gate is simulated to open fully within the failure time T_f computed by the empirical method at a chosen rate. It is easiest to use a linear rate of opening for the gate to simulate the piping hole, but one can make the gate open at any chosen rate by manually entering the gate opening versus time in the unsteady flow data.

The method for determining D is described below. Generally, it consists of two algebraic equations and two unknowns: the piping-hole height/width (D) and the length of the piping-hole (L) along the hole's center. L is inversely proportional to D for a given V_{er}. The mathematics involved follows with a sketch of a dam cross-section for clarification:

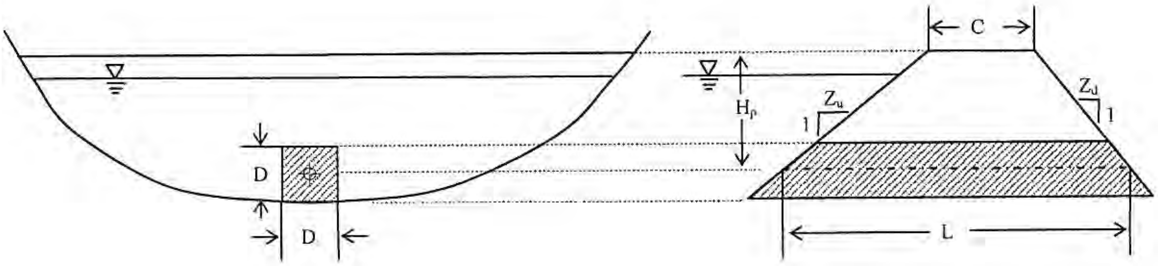


Figure 4 - Piping Hole Definition Sketch

The following equations can be derived from this geometry:

$$H_p(\textit{height from crest to center of piping hole}) = H_b - \frac{D}{2}$$

$$Z_t(\textit{sum of all side slopes}) = Z_u + Z_d$$

$$L\ (\textit{length of piping hole}) = H_p Z_t + C \rightarrow L = (H_b - \tfrac{D}{2})Z_t + C$$

$$V_{er}(\textit{volume eroded in cubic feet}) = D^2 L$$

Because L can be represented as a function of D and the dam geometry, substitution yields:

$$V_{er} = D^2\left(\left(H_b - \tfrac{D}{2}\right)Z_t + C\right)$$

$$V_{er} = D^2\left(H_b Z_t - \tfrac{DZ_t}{2} + C\right)$$

Reducing this equation yields:

$$V_{er} = (D^2 H_b Z_t) - \left(\frac{D^3 Z_t}{2}\right) + (D^2 C)$$

Separating and grouping by the power yields the final equation with only one unknown, D:

$$V_{er} = D^2(H_b Z_t + C) - \frac{D^3 Z_t}{2}$$

This cubic equation is difficult to solve explicitly, so an EXCEL look-up table was formulated to solve for D, which is available from the Colorado Dam Safety Branch with an example.

The above method yields the most accurate theoretical piping hole size for the empirical volume eroded. However, an approximate hole size can be calculated easily assuming $H_p = 0.7 \times H_b$ and then determining L and D from that with the basic geometry of the dam. This value of H_p was found to be close to the theoretical H_p for several trials made with different dams.

7.1.1.2 Spreadsheets

Two spreadsheets have been developed to assist the user with dam breach parameter estimation using the empirical methods recommended above. In addition to calculating breach parameters, both spreadsheets include an estimate of peak breach discharge, which is intended for use at the screening level of analysis. The equation used to estimate peak breach discharge was developed for the Simplified DAMBREAK program (Wetmore & Fread, 1984). This equation is discussed in more detail in Section 7.2.1 below. Because the equation is based on the breach geometry, it will provide different results depending on the empirical method chosen. Both spreadsheets include calculations to validate the results based on the discussion in Section 7.1.1 above.

Macdonald & Langridge-Monopolis and Washington State Spreadsheet

The Macdonald and Langridge-Monopolis (1984) calculates dam breach parameters based on the Breach Formation Factor (BFF). The BFF, which is product of reservoir storage volume in acre feet and reservoir depth in feet, is used to calculate the volume of material eroded from the embankment (V_{er}) during breach development. Breach dimensions are then calculated based on the geometry of the dam. The work of Macdonald and Langridge-Monopolis was later refined by Washington State (2007). The Washington State guidelines proposed new relationships for calculating the eroded volume and breach formation times, depending on the erosion resistance of the material used to construct the embankment.

Based on testing the proposed relationships on a wide variety of embankments, the spreadsheet calculations were selected using the preferred relationships as indicated in Table 3. The breach development times (T_f) for all cases are calculated using the relationships proposed by Washington State (2007). Table 4 below summarizes the equations used in the spreadsheet based on the type of embankment.

Table 4 – Summary of Macdonald & Langridge-Monopolis and Washington State Spreadsheet Calculations by Embankment Type

Embankment Type	Calculation of Embankment Volume Eroded (V_{er})	Breach Development Time (T_f)	Reference
Earthen (Cohesive)	$V_{er} = 3.264(BFF)^{0.77}$	$T_f = 0.036\, V_{er}^{0.36}$	Macdonald and Langridge-Monopolis (1984) & Washington State (2007)
Earthen (Cohesionless)	$V_{er} = 3.75(BFF)^{0.77}$	$T_f = 0.02\, V_{er}^{0.36}$	Washington State (2007)
Rockfill	$V_{er} = 0.714(BFF)^{0.852}$	$T_f = 0.02\, V_{er}^{0.36}$	Macdonald and Langridge-Monopolis (1984) & Washington State (2007)

As discussed in Section 7.1.1 above, a piping failure may result in reservoir evacuation prior to full breach development. To address this failure mode, the spreadsheet includes a feature to calculate piping hole dimensions. The size of the piping hole, which is assumed to be square, is calculated based on the embankment volume eroded (V_{er}) and the dam geometry. The peak discharge through the piping hole is then calculated using the orifice equation. The calculation assumes that the piping hole forms instantaneously by applying the head of a full reservoir. This conservative assumption is considered adequate for a screening level analysis.

Froehlich 2008 Spreadsheet

The Froehlich 2008 spreadsheet was developed according to the relationships proposed by Froehlich (2008). Using this method, breach dimensions are dependant only on the depth and volume of water stored by the dam. This method does not consider dam geometry or the type of soil used to construct the dam. The average breach width (B_{avg}) and failure time (T_f) are calculated as:

$$B_{avg} = 8.239 K_o V_w^{0.32} H_b^{0.04}$$

$$T_f = 3.664 \sqrt{\frac{V_w}{g H_b^{\,2}}}$$

Where:

K_o = Failure Mode Factor
H_b = Height of breach in feet
V_w = Reservoir volume stored in acre-feet

The spreadsheet automatically selects the K_o value based on the user-selected failure mode. The values are 1.0 and 1.3 for piping and overtopping failures, respectively. The spreadsheet allows the user to input the breach side slope ratio, but it should be noted that Froehlich recommended values of 0.7 and 1.0 for piping and overtopping failures, respectively.

7.1.2 Physically Based Models

NWS BREACH is currently the most widely used physically based model that can be used to estimate dam breach parameters. Based on Colorado Dam Safety Branch research into the BREACH program for numerous case studies (see Appendix A), the following potential problems have been identified and should be considered when using BREACH to estimate dam breach parameters:

1. Back-calculation of the piping orifice coefficient from BREACH output runs indicate the program may over-estimate this coefficient within BREACH vs. hand-calculated values based on equations developed by Fread (1988b). This problem appears to result in an over-estimation of breach flows for some Small and Minor dams with low storage intensities and an under-estimation of breach flows for dams with higher storage intensities.
2. The program causes the transition from pipe to weir flow to occur when the reservoir level reaches one-half of the pipe height above the top of the pipe. In other words if the piping hole height is 10 feet, then the crest collapses when the reservoir is 5 feet above the top of the piping hole. Based on observed dam failures of minor sized dams, this appears to force the collapse to occur prematurely. When combined with the high piping orifice coefficient, this issue may tend to drain the reservoir too rapidly and result in a smaller final breach configuration (less conservative).
3. After the crest collapses, the breach section gradually erodes laterally until the reservoir is drained enough to halt additional erosion. BREACH does not consider the head-cutting potential, so the lateral erosion may be overly simplified and the erosion rate is slow during this portion of the simulation. This may tend to make the total failure time long (less conservative).
4. The modeling algorithm for an overtopping failure erodes through the downstream slope and crest at the same grade as the downstream embankment slope using a sediment transport equation. Once the crest is eroded, the program starts eroding downward through the upstream slope, which, at the beginning erodes very rapidly straight down to the bottom of the dam without widening. Once the breach is cut through the dam, the program widens the breach at a slow rate. This algorithm ignores the head-cutting erosion process that actually occurs during an overtopping failure and results in a final breach configuration that may tend to be narrow (less conservative).

These limitations should be taken into account when BREACH is used for performing hazard evaluations. BREACH appears to be most applicable for Small or Minor dams with low storage intensities since the alternative methods (empirical equations) sometimes yield very small breach dimensions and failure times. Acceptance of this model for hazard classification studies will be allowed with reasonable justification. The results must be validated with the other recommended methods.

7.2 Breach Peak Discharge Estimation

7.2.1 Empirical Methods

Equations for breach peak discharge estimates were developed for both the MacDonald & Langridge-Monopolis (1984) and Froehlich (2008) methods. Wahl evaluated these equations by comparing predicted peak discharges to actual peak discharges and found significant scatter between observed data and that predicted by the equations. The Froehlich equation had the best correlation, but still could significantly over-predict or under predict the peak flow. The MacDonald & Langridge-Monopolis method is an outlier curve with significant scatter and appears to greatly over predict the peak flow. In several analyses performed with this guideline, it was determined that the MacDonald & Langridge-Monopolis equation produced peak flows significantly greater than that produced by an instantaneous failure to the ultimate breach geometry. In other words, the computed peak discharge using a weir flow equation with the final breach configuration and the reservoir level at H_w was less than that produced by the MacDonald & Langridge-Monopolis peak discharge equation, which is impossible unless the reservoir is infinitely large.

Wetmore and Fread (1984) provide an alternative to the MacDonald & Langridge-Monopolis (1984) and Froehlich (2008) equations for breach peak discharge. This equation was developed as part of the Simplified DAMBRK program (SMPDBK). It is essentially a weir equation of an instantaneous failure with a reduction factor. This reduction factor is dependent upon the reservoir surface area at full storage, the failure time, and H_w. As the size of the reservoir increases, this equation appropriately approaches that of an instantaneous failure and the peak flow will never exceed the instantaneous failure value. Because of the weir flow component of this equation, it is more physically based than a pure empirical equation. The breach dimensions can be determined with empirical models, and then those dimensions can be input into this equation to determine a predicted peak discharge. The equation is as follows:

$$Q_p = 3.1 B_{avg} H_w^{1.5} \left(\frac{\gamma}{\gamma + T_f \sqrt{H_w}} \right)^3$$

Where:

Q_p = Dam break peak discharge in cfs
B_{avg} = Average breach width in feet
H_w = Maximum depth of water stored behind the breach in feet
T_f = Breach development time in hours
γ = Instantaneous flow reduction factor = 23.4 A_s/B_{avg} (equivalent to 'C' in Wetmore and Fread (1984))
A_s = Surface area of the reservoir in acres corresponding to H_w

In several of the case studies analyzed in the preparation of this guideline, the predicted Q_p using the SMPDBK equation was greater than the actual computed peak flows using HECRAS, but the difference was marginal. As such, the SMPDBK Peak Flow Equation tends to produce reasonably conservative results. These equations provide only peak discharge values as opposed to a hydrograph. In cases where routing of the flow is not considered or predicted empirically, this equation can be used as part of a Screening level analysis and can indicate if a more sophisticated analysis is needed. For instance, if the SMPDBK equation produces a peak discharge that shows the critical structure is clearly not inundated, then the dam can be rated as Low Hazard and further work to determine a High or Significant Hazard rating is not warranted. If the estimated peak discharge from the SMPDBK equation clearly inundates the structure, then the dam should be rated as either High or Significant Hazard unless a more sophisticated analysis shows that a lower hazard class is appropriate.

In cases where the piping failure mode is not expected to progress to a full breach, the weir flow assumption of the SMPDBK equation above does not apply. In this case, a theoretical maximum breach discharge can be calculated with the orifice equation assuming that the piping hole opens to its maximum dimensions instantaneously:

$$Q_p = C_p D^2 \sqrt{2g \left(H_w - \frac{D}{2} \right)}$$

Where:

Q_p = Dam break peak discharge in cfs
C_p = Piping Orifice Coefficient
H_w = Maximum depth of water stored behind the breach in feet
D = Dimension of square breach hole

7.2.2 Parametric Models

7.2.2.1 Hydrologic Models

Some hydrology models, including HEC-1 and HEC-HMS, include a parametric dam breach algorithm. Breach parameters must be obtained using other methods and provided as input in the analysis including failure mode (piping or overtopping), breach bottom width, breach side slopes, and breach development time. A breach progression method must also be selected from the list that includes: linear, sine wave, or user defined. In addition, HEC-HMS requires a piping orifice coefficient and starting elevation for the piping failure mode and a weir coefficient for the overtopping failure mode. The HEC models simulate a failure by enlarging a trapezoid-shaped breach with time in accordance with the specific progression and geometry. The reservoir is drained through the breach opening in the duration specified by the development time using the Modified Puls method, thus producing a breach outflow hydrograph.

A review of the methodology and results of several case studies (see Appendix A) reveals that this method is adequate for most overtopping failure simulations, but issues regarding the piping mode failure simulation were noted. Review of the HEC-HMS manual indicates that a piping failure is simulated by expanding the piping hole radially outward around the starting elevation until the top of the circle reaches the dam crest, at which time the breach transitions to a trapezoidal shape and continues progressing. Flow through the circular opening is modeled as orifice flow, while the second-stage trapezoidal shape is modeled as a weir. This modeling algorithm raises several concerns that should be considered when using the piping failure mode in HEC-HMS:

1. Changing from a circular shape to a trapezoidal shape creates a discontinuity with the two different methods and results in irregularities in the outflow hydrograph.
2. Having the radial piping hole expand all the way to the dam crest using the orifice flow equation would mean that the reservoir level drops below the top of the orifice (pipe) for a certain amount of time while the orifice equation is still being used. This scenario would be modeled more accurately if the algorithm switched to the weir equation as soon as the water surface dropped below the top of the pipe. The HMS algorithm causes irregularities in the outflow hydrograph including a sudden drop in the flow when the reservoir level drops below the top of the pipe and a sharp increase when the model changes from orifice flow to weir flow.
3. Review of the case study results along with back-calculations show that the piping head used in the orifice flow portion of the model is measured from the designated starting piping elevation for all intervals regardless of the starting reservoir elevation. This situation would be modeled more accurately if the piping head were measured from the center of the piping hole at all times. If the starting piping elevation is set at the bottom of the dam, then the piping head is measured from the bottom of the piping hole as the hole only expands upward radially and the bottom half of the piping hole is ignored (because it is underground).

The first two issues described above only cause discontinuities in the shape of the hydrograph and would probably not have a significant impact on a hazard evaluation. However, the third issue, with the starting elevation set at the bottom of the dam, tends to cause an unrealistic increase in flow through the piping portion of the failure by overestimating the piping head as the hole expands. This tends to create a very sharp and high peak outflow within the piping portion of the failure. It also tends to cause a more rapid decrease in the reservoir level with higher flows occurring during the piping stage of the failure. As such, the peak of the breach hydrograph usually occurs during the piping mode of failure as opposed to during the weir flow mode. **To resolve this issue when modeling a piping failure breach with HEC-HMS, it is recommended that the starting piping elevation should always be set at the mid-point of the final breach height.** This will ensure that the head is always measured from the center of the pipe/dam. A comparison of HEC-HMS results (with the starting elevation set at the center elevation of the reservoir) to HEC-RAS results (with a piping failure starting at the bottom of the reservoir) showed similar peaks and time to peaks in the resulting breach hydrographs.

The HEC-1 model simulates a dam breach by assuming weir flow through a trapezoidal section that progresses linearly from no breach at the top of the dam to the specified final parameters at the bottom of the breach in the time T_f. The piping portion of a failure is not considered and the only progression available is linear. Therefore, the results may not be valid for a piping failure of a smaller reservoir when the piping portion may be significant enough to impact the final hydrograph.

Due to the above issues, caution should be exercised when using HEC-1 and HEC-HMS to simulate a piping failure, especially when the final results at a critical location downstream indicate a borderline situation between hazard ratings. However, because of their simplicity and ease of use, both models are valuable for simulating an overtopping breach for a simple or intermediate analysis. Also, for a simple or intermediate analysis of a piping failure, HEC-1 can be useful for a larger reservoir when the piping portion of the failure is not as significant and HEC-HMS can be useful if the starting piping elevation is set at mid-height of the reservoir behind dam.

7.2.2.2 Hydraulic Models

The latest versions of the HEC-RAS model include algorithms to model both overtopping and piping breaches. HEC-RAS uses hydraulic principles through cross sections upstream and downstream of the dam to define how the reservoir drains during the formation of a dam breach. The dam crest is modeled as an inline weir and either a piping failure or overtopping failure is simulated with enlargement of the breach occurring over time as defined by a specified breach progression. Flow through the piping hole is calculated as orifice flow and flow through the breach is calculated as weir flow. The water surface profile upstream of the dam is back-calculated using unsteady momentum and hydraulic principles for each time step and the resulting drawdown through the hole and/or breach produces an outflow hydrograph. Resulting water levels for each time step downstream of the dam are used to model potential backwater effects and the weir and orifice coefficients are automatically adjusted for submergence, if necessary. HEC-RAS can also model a piping failure that does not progress to the point of collapsing the crest. In this scenario, the piping hole is simulated as a sluice gate.

Compared to HEC-HMS, the HEC-RAS program models a dam failure, especially a piping failure, more correctly and accurately for the following reasons:

1. Modeling a dam failure using hydraulic principles is usually more accurate than a hydrologic model because the modeler can more accurately simulate the shape of the reservoir, tailwater effects, and drawdown effects. Put simply, a dam failure is more accurately defined as a hydraulic process than a hydrologic one. HEC-RAS has the capability to model the pipe with an initial piping elevation set at the bottom of the dam (most piping failure situations); the piping hole is modeled as a rectangular hole, which is more consistent with the final trapezoidal shaped breach section, thereby reducing discontinuity. The bottom width of the hole enlarges proportionally to the final bottom width according to the selected progression, as does the height of the hole toward the final breach depth. This will make the hole height/width ratio greater than one if the final breach parameters chosen show

a bottom width narrower than the dam height, but since the orifice flow is based upon the area of the orifice and not the width versus the height, this is a valid assumption.

2. Once the water level drains down to the top of the enlarging piping hole, the crest is assumed to collapse and the algorithm transitions to weir flow. The bottom width and the top width of the breach continue to enlarge laterally until the final defined width and side slopes are reached. Unlike HEC-HMS, HEC-RAS never assumes orifice flow with a reservoir level below the top of the piping hole. It should be noted that in several modeled case studies, the crest collapses when the height of the piping hole reaches near $0.6H_b$ and the peak flow occurred in the weir flow portion of the failure soon after the crest collapses. This is dependent on the drawdown rate versus the breach progression, and selected final parameters, but helps support the reasonableness of the program.
3. Unlike HEC-HMS, the piping head used in orifice flow is measured from the center of the piping hole at all times regardless of its location and size.
4. One cannot model a non-breaching piping failure in HEC-HMS. This mode of failure occurs when the reservoir volume relative to the dam's size (storage intensity – SI) is small enough that the reservoir would drain through the hole before the crest collapses and forms a breach. The specific situations when this would occur are discussed in detail in section 7.1.1.1, but is a potential situation for tall (say greater than 35') Small dams with a volume less than 100AF when SI is less than or equal to 2.5 and Minor dams when SI is less than or equal to 1.5. Instead of using the breach routine, HEC-RAS can be forced to model a progression of this mode of failure using a sluice gate and a square hole while manually entering the gate opening at a chosen rate to the failure time.

Unfortunately, with the more accurate HEC-RAS method, more time consuming and detailed input is required as follows:

1. Dam and spillway data are needed to define the dam as an inline weir. HEC-RAS version 4.0 includes the capability to model the dam crest weir flow and breach weir flow with different weir flow coefficients. A weir coefficient of 3.08 is appropriate for the breach section. A weir flow coefficient of 2.6 is appropriate for the dam crest.
2. HEC-RAS must have a base flow to perform an unsteady simulation. For on-stream reservoirs that are normally full and spilling, it is recommended to use an estimate of an average year spring runoff monthly flow extrapolated from nearby gages. As discussed above, the spillway needs to be modeled to pass this flow at the correct "normal" spring runoff elevation. For off stream reservoirs and/or when the reservoir is not normally above the emergency spillway crest, a fictitious outlet using the sluice gate option needs to be input to pass the base flow without exceeding the emergency spillway elevation. The pipe hole size may be calculated based upon this base flow using the orifice flow equation. In this situation, the base flow can be estimated from normal to peak runoff. The fictitious outlet and base flow may have to be adjusted if there are stability problems in the model.
3. Breach parameters, including the mode of failure (piping or overtopping), breach width, side slopes, and failure time are all similar to the HEC-HMS input data and need to be obtained from other sources. Also, like in HEC-HMS, a breach progression curve (either linear, sine wave, or a user defined curve) along with a piping orifice coefficient and starting elevation for a piping failure and a weir coefficient for overtopping must be specified. The starting water surface elevation may be specified anywhere between the bottom and top of the reservoir and HEC-RAS will enlarge the piping hole and compute the orifice flow. Although the most realistic starting piping elevation is usually around the outlet pipe, a sensitivity analysis sometimes reveals that a different piping elevation may produce the most "realistically conservative" result. Recommended options for the orifice and weir coefficients along with progressions are discussed later since these parameters are the same for both HEC-HMS and HEC-RAS.

Detailed instructions on how to build a HEC-RAS model is not within the scope of this document. However, a few "Rules of Thumb" for establishing a stable working model are provided as follows:

1. The model can simulate a reservoir as either a storage area with a defined stage-storage relationship, or as a series of cross-sections cut through the reservoir. The storage area method has the benefit of accurately modeling the actual storage within the reservoir, but it does not calculate hydraulic losses as water in upper portions of the reservoir travels to the dam breach.
2. The USACE HEC has produced a guidance document for modeling a dam failure with the storage area method. This document is provided in Appendix B (Gee, 2006).
3. As outlined in Appendix B, the HEC-RAS model must contain at least two cross-sections upstream of the dam. The furthest upstream section is connected to the storage area and the downstream section is associated with the dam which is modeled as an in-line structure. Adding a third section between the 2 sections upstream of the dam is also recommended to allow for a better solution of the unsteady flow analysis. In order to add the third section, the section at the upstream toe can be simply copied upstream one foot to provide the connection to the storage area. Then, one section can be interpolated half-way between to generate the third section. It is important to keep the distance between these sections small to prevent inadvertently increasing the storage of the reservoir.
4. Of the two sections upstream of the dam, the downstream section parallels the dam along its upstream toe. It can be defined from known bathymetric data, or it can be easily defined as a simple trapezoid with its top width coinciding with the length of the dam crest and at the crest elevation or slightly higher, and the bottom points coinciding with the toe of the dam at its maximum section. The bottom width of the trapezoid should be equal to or greater than the breach bottom width. This simple section is acceptable because it is not used for hydraulic calculations or to define the storage within the reservoir. The storage is defined by a user-input elevation-storage table.
5. The dam is defined by the crest profile input as an in-line structure. The crest profile is superimposed onto the cross-section immediately upstream of the dam. When simulating a dam breach, the breach geometry is cut into the crest profile with the specified breach geometry and at the user-defined rate.
6. It should also be noted that in HECRAS 4.0, it is necessary to set the bottom elevation of the cross-sections upstream of the dam at or above the minimum elevation of the stage-storage defining the reservoir storage. Any storage input below the minimum elevation of the breach will be treated as dead storage.
7. At least two sections downstream of the dam must be established to model the failure and address tailwater effects. The first one is at, or just downstream of, the dam's downstream toe. The lowest section is then established far enough downstream to help establish the downstream channel slope and tailwater submergence effects. Do not add any more sections downstream until the failure can be modeled satisfactorily.

If bathymetric data are available for an oddly-shaped reservoir (long and skinny or very wide), a more accurate way to model the breach would be to cut cross-sections through the bathymetric surface.

1. Define two cross sections upstream of the reservoir, with the second one being set at the high-water line of the reservoir. These sections do not define the reservoir so their configuration is not critical, but they are needed to start the model.
2. When utilizing the cross section method, the sections within the reservoir need to capture significant changes in the ground slope (if known) and shape of the reservoir. Set the overbank stations at the high-water line. The Manning n-values in the reservoir are usually low because of the reservoir depths and a drained reservoir is void of vegetation. However, for steeper and wide reservoirs, it may be necessary to increase the roughness in the basin for model stability as the

dam fails. It is also advisable to set the bottom of each reservoir cross-section as a point with no width slightly below the invert viewed on the contour lines to help with model stability.

3. The use of interpolated sections within the reservoir basin is usually not necessary. In fact, too many sections may cause stability problems. However, if certain segments of the reservoir are steep, then interpolated sections may be needed.
4. The downstream-most section in the reservoir (which defines the inline weir/dam) is obtained from a section along the upstream toe that parallels the dam. Like other sections in the reservoir, set the bottom of a point slightly lower than the breach bottom at its center for model stability. Since HEC-RAS does not recognize water downstream of this section that lies against the upstream slope, placing this section (distance-wise) half-way between the upstream toe and crest usually helps in establishing the correct volume. The distance to the crest, side slopes of the dam, and distance to the downstream section will have to be reduced to compensate, but this has no influence on the computations or results.

The DAMBRK model is mentioned here as an alternative to using HEC-RAS to model a dam failure. It has the capability to model overtopping and piping dam failures with an input of the final breach parameters similar to HEC-RAS. The BOSS version of DAMBRK is much more user friendly than the original NWS DAMBRK model. A significant drawback for using DAMBRK is the cross-sectional input requirements maybe over simplified for routing dam break floods downstream because it requires the same number of points at every cross section and the model seems to have more convergence problems than HEC-RAS. For very long reservoirs, where the hydraulics inside the reservoir basin during a dam failure may have a significant impact on the hydrograph, a different routine within the DAMBRK program allows for defining the reservoir with cross sections. However, due to the convergence and over simplification of the sections, it would probably be better to use HEC-RAS for this situation.

7.2.2.3 Parameters Common to Hydraulic and Hydrologic Models

Both hydraulic and hydrologic parametric models require additional input beyond the breach parameters discussed so far. These parameters include orifice flow coefficients, weir flow coefficients and breach progressions. Depending on the type of parametric model used (hydraulic or hydrologic) and the mode of failure being modeled (piping or overtopping) the recommendations for some of these parameters may change.

7.2.2.3.1 Orifice Coefficients (C_p)

The orifice coefficient defines flow through the piping hole before the models shift to weir flow as

$$Q = C_p A \sqrt{2gH_p}$$

Where:

H_p = Piping head (difference between the water level and the centroid elevation of the piping-hole)
C_p = Piping Orifice Coefficient
A = Cross sectional area of the piping hole.

Danny Fread, author of the BREACH program, outlines a method of computing the orifice coefficient, C_p, which is dependent on the material of the dam and length and size of the piping hole. This equation is based upon the Darcy friction factor (f) where:

$$f = 0.015 \left(\frac{D_{50}}{D}\right)^{0.167}$$

Where:

D_{50} = Average grain size in millimeters for the dam embankment material.
D = Piping hole width in feet (assumed to be square).

And:

$$C_p = \sqrt{\frac{1}{1 + \frac{fL}{D}}}$$

Where:

L = Length of the piping hole (in feet) along its centerline.

C_p varies from very small for a long L and small D value, as seen during the initial piping stages, to values approaching 1.0 for very large piping holes and shorter L values. C_p is always less than or equal to 1.0. C_p is also inversely proportional to the grain size, being larger for smaller D_{50} values. This approach seems reasonable, but unfortunately, it appears that BREACH does not use it correctly (see discussion under "Physically Based Models") and both HEC-RAS and HEC-HMS, use a constant C_p value. Using an average C_p from this equation in both HEC-RAS and HEC-HMS computed for various piping hole sizes appears reasonable and appropriate.

Using the orifice coefficient equation for various size dams and varying the potential piping hole sizes from a D = 0.1 ft to 0.6 H_b shows that C_p varies from 0.1 to 0.87. The average computed C_p values, while uniformly varying D for dams with coarse-grained material and fine-grained material, are shown in Table 5 below.

Table 5 - Recommended Orifice Piping Coefficients Based on Dam Height

Height of Dam, H_w (ft)	Average C_p for **coarse grained** soils (D_{50}>0.25 mm).	Average C_p for **fine grained** clayey soils (D_{50}<0.01 mm)
100	0.70	0.77
50	0.68	0.75
30	0.66	0.74
20	0.64	0.73
10	0.61	0.70

Table 5 shows that the average C_p is close to 0.7 for all typical dams and it is reasonable to assume this value for most analyses. As the height of the dam gets smaller, a C_p of 0.6 can be assumed; for very large dams with fine grained material, a C_p of 0.8 can be assumed. Table 5 is for typical dams with 3:1 upstream slope, 2:1 downstream slope, and corresponding typical dam crest widths. For borderline hazard ratings, when more detailed analysis is required and the dam has very coarse grained material or the dam cross sectional varies from the typical section, it may be necessary to use the aforementioned equation for different piping hole widths to calculate an average Cp. An EXCEL spreadsheet has been developed for calculating C_p and is available from the Colorado Dam Safety Branch. Note that choosing a high C_p is not necessarily more conservative than using a low C_p. In fact, in many cases, the opposite occurs. A high C_p will cause more water to drain through the piping hole before the transition to weir flow and the resulting peak during weir flow may actually be lower. This situation is dependent on several factors and there is no rule of thumb regarding choosing a conservative value for C_p. In some borderline cases, a sensitivity analysis of C_p may be warranted.

7.2.2.3.2 Weir Coefficients (C_w)

Piping Failures

After the dam crest collapses, the breach section erodes laterally and outflow through the breach is modeled as weir flow. During a piping mode, the configuration of the weir consists of a flat approach channel through the breach with a vertical drop-off to the tailwater section resulting from head-cutting during both the piping mode and weir flow. For free-flow conditions through the breach (no tailwater effects), critical depth at the drop-off becomes the control and the reservoir level upstream equals the corresponding energy grade line. Back-calculating a corresponding C_w for this situation, shows **C_w = 3.08.** This is supported by King and Brater's Handbook of Hydraulics, which states the maximum allowable weir coefficient for a broad-crested weir is 3.087. **The value of 3.08 is automatically used in HEC-HMS and it is recommended for use for all piping situations in HEC-RAS.**

Overtopping Failures

The weir coefficient is defined with the dam top for both HEC-HMS and HEC-RAS for an overtopping failure. It is not only used to define the flow through the breach during the failure, but is used to define the flow over the dam before failure. In reality, this situation would warrant two different weir flow coefficients, but that capability is not available.

During the overtopping phase before failure, the crest acts like a broad crested weir with side slopes. Extrapolating from King and Brater, C_w is usually around 2.6 to 2.8 for a typical dam, but can vary from 2.6 to 3.087 for a broad crested weir.

During the breach and after the crest and downstream slope have eroded severely, the crest width is reduced to almost zero and the downstream slope can approach vertical due to head-cutting. This would increase the free-flow C_w value to over 3.0. HEC-1 automatically uses 3.1 for an overtopping failure, which appears appropriate for modeling a dam failure assuming no crest and a vertical downstream slope. Note that HEC-RAS will automatically adjust this value for tailwater effects if the weir flow becomes submerged, so the free-flow C_w should always be specified.

If the user is using earlier versions of HECRAS where only one C_w can be specified, the flow during the breaching of the dam after erosion of the downstream slope and crest is more critical than the overtopping flow prior to breaching and it is best to use the C_w for the breaching portion. **A value of C_w = 3.08 should be used for overtopping failures.** If it is critical to maintain the water surface elevation overtopping the dam by the defined inflow, the crest profile length can be adjusted to hold the water surface at the same elevation as if a smaller C_w were used (i.e. adjust the dam length to control the computed water surface elevation in the reservoir). However, since HEC-RAS version 4.0 includes the capability to model the dam crest weir flow and breach weir flow with different C_w values, the crest profile length will not need to be adjusted.

7.2.2.3.3 Breach Progressions

Piping Failures

A breach progression must be specified for both HEC-HMS and HEC-RAS piping failure analyses. The breach progression defines the rate at which the piping hole or breach width enlarges with time. Both of these models allow the user to choose a linear progression, sinewave progression (starts out slowly, with a more rapid increase in the center of the progression and then ending more slowly), or a user defined (manual) progression. DAMBRK allows the user to choose a linear progression or exponential progression.

Generally, it has been observed that breaches progress slowly at the start and end of a failure and more rapidly in the middle of the progression. This leads one to believe that the sinewave progression most accurately models a breach. However, if the hydraulics downstream of the dam create enough tailwater to affect the outflow hydrograph, then a linear progression may be more appropriate if the model used cannot take submergence into account.

When using HEC-HMS, there was minimal difference in the results between the linear and sinewave progression methods for the cases studies evaluated for these guidelines and there was no consistency as to which progression produced the larger peak flow. It is hypothesized that this inconsistency is a result of the fact that HEC-HMS cannot account for tailwater effects. **It is therefore recommended to check both the linear and sinewave progressions with HEC-HMS and use the one that produces the most conservative results.**

More sensitivity to the selected breach progression was observed when using the HEC-RAS program for the case studies evaluated for these guidelines. The HEC-RAS results showed that the sinewave progression produced the higher peak flow in all cases. It is hypothesized that HEC-RAS's ability to account for the tailwater effects through its hydraulic calculations may contribute to the lower peak discharges calculated when the linear progression is used. Using the linear progression in HEC-RAS may be similar to accounting for the tailwater effects twice which results in a less conservative peak discharge. **Because of this, the sinewave progression is recommended for use in HEC-RAS models using parameters based upon empirical methods.**

Overtopping Failures

Like with piping failures, a breach progression must be specified for both HEC-HMS and HEC-RAS analyses of overtopping failures. Overtopping failures do not have the transition from orifice to weir flow with a crest collapse that a piping failure requires. The breach progression is therefore not as variable and a more uniform progression rate is applicable. In an overtopping failure, the failure time is assumed to start when the upstream slope starts eroding. This may not happen until a significant time period after the dam starts to overtop. Both the linear and sinewave progressions are appropriate in this situation.

For both HEC-HMS and HEC-RAS, there was not much difference in the results between the two progression methods for the one overtopping case-study analyzed. HEC-HMS produced peak flows that were within 8% of each other and the HEC-RAS results were within 1%. As discussed above, **it is recommended to check both methods and use the most conservative results (i.e. highest peak discharge).** Tailwater factors may also have an impact and should be considered where appropriate.

List of Variables

(See Figures 1&2)

$\boldsymbol{H_b}$ = Height of breach in feet, which is the vertical distance between the dam crest and breach invert.

$\boldsymbol{H_w}$ = Maximum depth of water stored behind the breach in feet (usually depth from emergency spillway crest down to breach invert for a full, fair-weather breach)

$\boldsymbol{V_w}$ = Reservoir volume stored corresponding to H_w in acre-feet (AF)

BFF (Breach Formation Factor) = H_wV_w in acre-feet^2 – used for MacDonald & Langridge-Monopolis and Washington State methods only.

$\boldsymbol{V_{er}}$ = Volume of dam eroded in cubic yards during a breach. Used for MacDonald & Langridge-Monopolis and Washington State methods only. This is the same as $B_{avg}W_{avg}$ for a full breach or D^2L for a piping only failure (variables defined below).

$\boldsymbol{B_{avg}}$ = Average breach width in feet. For a trapezoidal section, this is the width of the breach at the mid-point, $H_b/2$.

$\boldsymbol{Z_b}$ = Side slopes of breach (Z_b Horizontal: 1 Vertical).

$\boldsymbol{Z_d}$ = slopes of downstream face of the embankment (Z_d Horizontal: 1 Vertical).

$\boldsymbol{Z_u}$ = slope of the upstream face of the embankment (Z_u Horizontal: 1 Vertical).

$\boldsymbol{Z_t}$ = sum of the upstream and downstream embankment slopes, $Z_u + Z_d$

$\boldsymbol{B_b}$ = breach bottom width in feet: $B_{avg} - H_bZ_b$

$\boldsymbol{W_{avg}}$ = Average width of dam in direction of flow (feet). This is the width at the mid-point of H_b: $W_{avg} = C + H_b\frac{(Z_u+Z_d)}{2}$

$\boldsymbol{T_f}$ = breach development time in hours.

$\boldsymbol{C}$ = width of the dam crest in feet.

$\boldsymbol{g}$ = acceleration due to gravity, which equals 32.2 feet/sec^2

SI= Storage Intensity = V_w/H_w acre-feet/foot

ER = Erosion Rate = B_{avg}/T_f feet/hour

$\boldsymbol{L}$ = Length of piping hole, feet

$\boldsymbol{D}$ = Piping hole height/width (assumed square), feet

$\boldsymbol{H_p}$ = Height from center of piping hole to dam crest = $H_b - \frac{D}{2}$

$\boldsymbol{A_s}$ = Surface area of reservoir (acres) at reservoir level corresponding to H_w

$\boldsymbol{Q}$ = Discharge in cfs

$\boldsymbol{Q_p}$ = Peak dam break discharge at the dam in cfs

$\boldsymbol{Q_r}$ = Routed peak discharge in cfs at a certain distance, X, downstream of the dam

$\boldsymbol{X}$ = Distance downstream from the dam along the floodplain in miles

$\boldsymbol{D_{50}}$ = Mean soil particle diameter in millimeters

$\boldsymbol{A}$ = Area of the piping hole in square feet: D^2

$\boldsymbol{C_p}$ = Piping orifice coefficient

$\boldsymbol{C_w}$ = Weir coefficient

$\boldsymbol{f}$= Darcy friction factor

γ = Instantaneous flow reduction factor = 23.4 A_s/B_{avg}

$\boldsymbol{K_o}$ = Froehlich Failure Mode Factor

Attachment C
Steep Slope Riprap Sizing per the Gravel Guidelines

Wright Water Engineers, Inc.

CALCULATION SHEET

Project: UDFCD Gravel Mining Criteria
Job. No.: 121-030.000
Date: 10/22/2012
Subject: Steep Slope Riprap Sizing

Design: JMN
Check: TAE

I. Purpose

Calculate riprap size and layer thickness

II. References and Assumptions

See attached hand written calculation sheet Pitside riprap sample calculation, dated 10-17-12

III. Calculations

	2.5:1		3:1		4:1		5:1	
	1,000 cfs	20,000 cfs	1,000 cfs	20,000 cfs	1,000 cfs	20,000 cfs	1,000 cfs	20,000 cfs
SF =	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
γ =	62.4	62.4	62.4	62.4	62.4	62.4	62.4	62.4
S_s =	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
ϕ =	41	41	41	41	41	41	41	41
S =	0.4	0.4	0.33	0.33	0.25	0.25	0.2	0.2
R =	0.2	0.5	0.19	0.58	0.2	0.61	0.21	0.64
θ =	21.8	21.8	18.4	18.4	14	14	11.3	11.3
D_{50} (ft) =	3.5	8.9	2.2	6.8	1.4	4.3	1.1	3.2
D_{50} (in) =	43	107	27	82	17	52	13	39

AE: Trend in riprap sizing appears reasonable. Massive size requirements for steep slopes and high flows.

WRIGHT WATER ENGINEERS, INC.
2490 West 26th Ave., Suite 100-A
Denver, Colorado 80211
Tel. (303) 480-1700

Date 10-17-12 Sheet 1 of 4
Proj. No. 121 030 000
Proj. Name UDFCD Gravel Mining
Des. By JMN Ckd. By

Subject Pitside riprap sample calc

1) Size Riprap per Gravel Mining Guidelines for embankments

2) Reference
   a) UDFCD ... Guidelines for Gravel Mining ... April 2004
   b) FHWA Hydraulic Toolbox software
   c) UDFCD USDCM

3) Assumptions
   a) slope = 2.5:1
   b) SF = 1.25
   c)

2.4.1 Riprap. When riprap is used to stabilize the pitside slope, the rock shall be placed at a slope no steeper than 2.5H: 1V. Riprap sizing will be in accordance with the safety factor method for flow on a plane sloping bed. The minimum safety factor (*SF*) shall be 1.25 (if *SF* = 1, riprap is at condition of incipient motion.

$$SF = \frac{\cos\theta \tan\phi}{n \tan\phi + \sin\theta}$$

$$n = \frac{21\,\tau_s}{(S_s - 1)\,\gamma\, D_{50}}$$

$$\tau_s = \gamma R S$$

In which,

θ	=	face slope of pitside bank, in degrees to the horizontal
ϕ	=	angle of repose of pitside bank construction materials in degrees
n	=	stability factor
S_s	=	specific gravity of riprap particles
γ	=	specific weight of water = 62.4 lbs/ft3
D_{50}	=	median riprap particle size, in feet
R	=	hydraulic radius at normal depth of flow down pitside slope, in feet
S	=	face slope of pitside bank, in feet per foot
SF	=	Safety factor
τ_s	=	Tractive force

WRIGHT WATER ENGINEERS, INC.
2490 West 26th Ave., Suite 100-A
Denver, Colorado 80211
Tel. (303) 480-1700

Proj. No. 121 030 000
Proj. Name UDFCD Gravel Mining
Des. By JMN Ckd. By
Subject Pitside Riprap Sample Calc

4) Calculation

$$SF = \frac{\cos\theta \tan\phi}{\eta \tan\phi + \sin\theta}$$

$$\eta\, SF \tan\phi + SF \sin\theta = \cos\theta \tan\phi$$

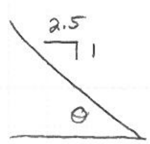
$$\eta = \frac{\cos\theta \tan\phi - SF \sin\theta}{SF \tan\phi}$$

$$\eta = \frac{21 \tau_s}{(S_s - 1)\gamma D_{50}}$$

$$D_{50} = \frac{21 \tau_s}{(S_s-1)\gamma \eta} \qquad \tau_s = \gamma R S$$

$$D_{50} = \frac{21 \gamma R S}{(S_s - 1)\gamma \left[\dfrac{\cos\theta \tan\phi - SF \sin\theta}{SF \tan\phi}\right]}$$

$\theta \rightarrow$



$\tan\theta = \frac{1}{2.5}$ $\quad\theta = 21.8°$

$\phi \rightarrow$ angle of repose, use 41° to start (approx. point of convergence Chow fig 7-10 & FHWA HEC 11 ('89) fig 25)

$S = 0.4$ ft/ft

$S_s = 2.5$ (typ min allowed in CDOT regs)

$R = 0.2$ ft

WRIGHT WATER ENGINEERS, INC.
2490 West 26th Ave., Suite 100-A
Denver, Colorado 80211
Tel. (303) 480-1700

Date 10-17-12 Sheet 3 of 4
Proj. No. 121 030 000
Proj. Name UDFCD Gravel Mining
Des. By JMN Ckd. By

Subject Pitside Riprap Sample Calc

$$D_{50} = \frac{21\,(62.4\ \#/ft^3)\,(0.2\ ft)\,(0.4\ ft/ft)}{(2.5-1)\,(62.4\ \#/ft^3)\left[\frac{\cos 22^\circ \tan 41^\circ - 1.25 \sin 22^\circ}{1.25 \tan 41^\circ}\right]}$$

$$\boxed{D_{50} = 3.6\ ft \quad \text{or} \quad 44\ in}$$

if $R = 0.5\ ft$

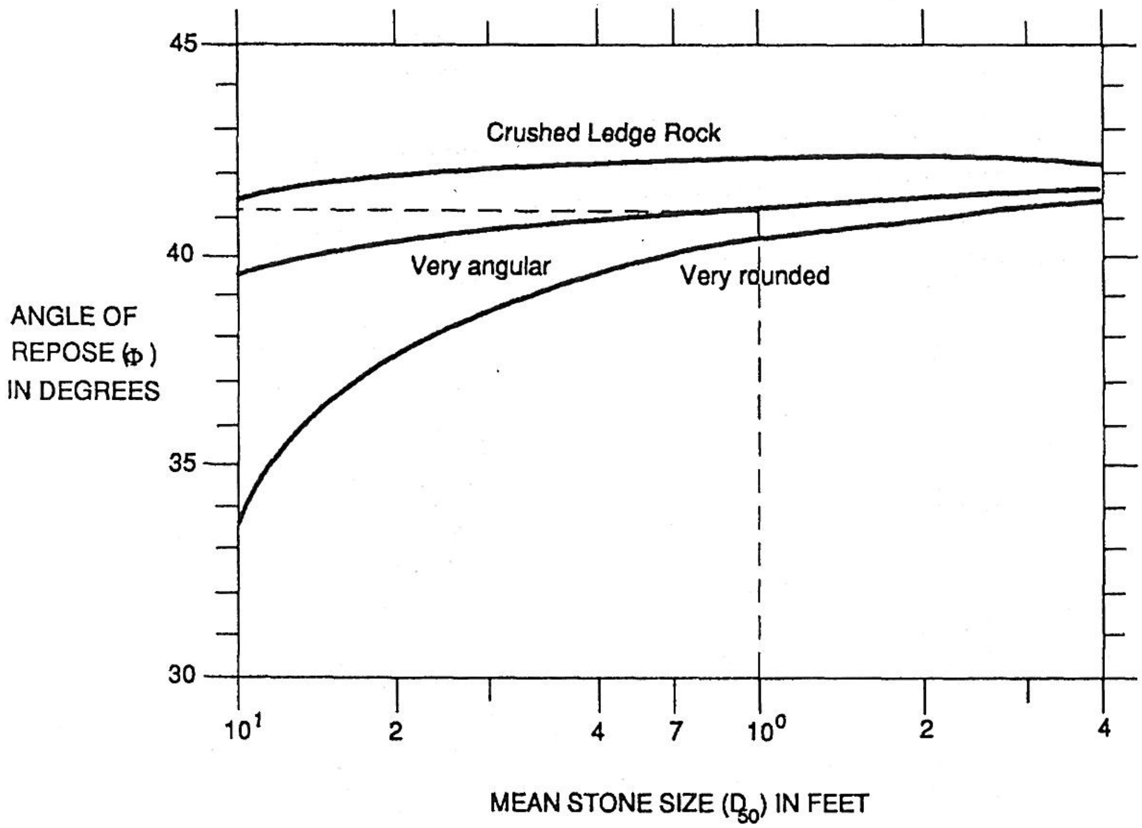
$D_{50} = 9\ ft$ or $108\ in$

WRIGHT WATER ENGINEERS, INC.
2490 West 26th Ave., Suite 100-A
Denver, Colorado 80211
Tel. (303) 480-1700

- run FHWA hydraulic radii w/ slopes of 3:1, 4:1, 5:1 and
  Q = 1,000 cfs or 20,000 cfs

Slope	1,000 cfs R	20,000 cfs R	θ
3:1 (.33 ft/ft)	0.19	0.58	18.4°
4:1 (.25 ft/ft)	0.2	0.61	14°
5:1 (.2 ft/ft)	0.21	0.64	11.3°

see spreadsheet calcs for multiple calc scenarios
w/ 2.5:1 run for back checking numbers



Example 1. D_{50}=1.0 ft; Angular Riprap

$\Phi = 4.1^{\circ}$

Figure 25. Angle of repose in terms of mean size and shape of stone (chart 3); example 1.

U.S. Department
of Transportation

**Federal Highway
Administration**

Hydraulic Engineering Circular No. 11

Publication No. FHWA-IP-89-016
March 1989

Design of Riprap Revetment

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

Hydraulic Analysis Report

Project Data

Project Title: UDFCD Gravel Mining Criteria / WWE Job No. 121-030.000

Designer: WWE JMN

Project Date: Tuesday, October 09, 2012

Project Units: U.S. Customary Units

Notes:

Channel Analysis: Channel Analysis 20000

Notes: For hydraulic radius required in pitside riprap size calculations, assume a 100-feet wide trapezoidal channel with less than 1% side slopes to mimic sheet flow down the slope. Run 1,000 cfs and 20,000 cfs to establish a range of hydraulic radii.

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 1000.0000 (ft/ft)

Side Slope 2 (Z2): 1000.0000 (ft/ft)

Channel Width: 100.0000 (ft)

Longitudinal Slope: 0.4000 (ft/ft)

Manning's n: 0.0400

Flow: 20000.0000 (cfs)

Result Parameters

Depth: 1.0709 (ft)

Area of Flow: 1254.0232 (ft^2)

Wetted Perimeter: 2241.8960 (ft)

Hydraulic Radius: 0.5594 (ft)

Average Velocity: 15.9487 (ft/s)

Top Width: 2241.8949 (ft)

Froude Number: 3.7580

Critical Depth: 1.8524 (ft)

Critical Velocity: 5.5301 (ft/s)

Critical Slope: 0.0237 (ft/ft)

Critical Top Width: 3804.7833 (ft)

Calculated Max Shear Stress: 26.7308 (lb/ft^2)

Calculated Avg Shear Stress: 13.9616 (lb/ft^2)

Channel Analysis: Channel Analysis 1000

Notes:

Input Parameters

Channel Type: Trapezoidal

Side Slope 1 (Z1): 1000.0000 (ft/ft)

Side Slope 2 (Z2): 1000.0000 (ft/ft)

Channel Width: 100.0000 (ft)

Longitudinal Slope: 0.4000 (ft/ft)

Manning's n: 0.0400

Flow: 1000.0000 (cfs)

Result Parameters

Depth: 0.3185 (ft)

Area of Flow: 133.3073 (ft^2)

Wetted Perimeter: 737.0412 (ft)

Hydraulic Radius: 0.1809 (ft)

Average Velocity: 7.5015 (ft/s)

Top Width: 737.0409 (ft)

Froude Number: 3.1084

Critical Depth: 0.5262 (ft)

Critical Velocity: 3.0348 (ft/s)

Critical Slope: 0.0354 (ft/ft)

Critical Top Width: 1152.4085 (ft)

Calculated Max Shear Stress: 7.9503 (lb/ft^2)

Calculated Avg Shear Stress: 4.5145 (lb/ft^2)

Attachment D
Steep Slope Riprap Sizing per Hydrologic Engineering Circular (HEC) 23

Wright Water Engineers, Inc.

CALCULATION SHEET

Project: UDFCD Gravel Mining Criteria
Job. No.: 121-030.000
Date: 10/22/2012
Subject: Steep Slope Riprap Sizing

Design: JMN
Check: TAE

I. Purpose

Calculate riprap size and layer thickness

II. References

1. Bridge Scour and Stream Instability Countermeasures: Vol .2 FHWA HEC 23, Sept. 2009
2. FHAD South Platte River, Adams County, CO., UDFCD/CDM April 2005
3. Technical Review Guidelines for Gravel Mining Activities Within or Adjacent to 100-Year Floodplains,

III. Assumptions and Equations

1. Overtopping flow rates: $Q_1 =$ 1,000 cfs
   $Q_2 =$ 20,000 cfs
2. Embankment Length = 1,000 ft
3. Interstitial Velocity =

$$\boxed{V_i = 2.48\sqrt{g d_{50}}\left(\frac{S^{0.58}}{C_u^{2.22}}\right)}$$

Ref. 1 eqn. 5.1

where:
- $V_i =$ Interstitial Velocity (ft/s)
- $g =$ Acceleration due to gravity (32.2 ft/s^2)
- $d_{50} =$ Particle size for which 50% is finer by weight (ft)
- $C_u =$ Riprap Coefficient of uniformity (d_{60}/d_{10})
- $S =$ Embankment slope (ft/ft)

4. Riprap size =

$$\boxed{d_{50} = \frac{K_u q_f^{0.52}}{C_u^{0.25} S^{0.75}}\left[\frac{\sin\alpha}{(S_g\cos\alpha - 1)(\cos\alpha\tan\phi - \sin\alpha)}\right]^{1.11}}$$

Ref. 1 eqn. 5.2

where:
- $d_{50} =$ Particle size for which 50% is finer by weight (ft)
- $K_u =$ 0.525 constant (English)
- $q_f =$ Unit discharge at failure (cfs/ft)
- $C_u =$ 2.1 Riprap Coefficient of uniformity (d_{60}/d_{10})
- $S =$ Embankment slope (ft/ft)
- $S_g =$ 2.5 Riprap specific gravity
- $\alpha =$ Embankment slope (degrees)
- $\varphi =$ Riprap angle of repose (41°) 0.73

5. Weir Equation

   $H = (Q/CL)^{0.67}$ broadcrested weir w/ C = 2.84

6. Average velocity

   $V_{avg} = \eta V_i$ $\eta =$ riprap porosity

7. Interstitial avg. flow depth

   $y = q_f / V_{avg}$

IV. Calculations

1. 1,000 cfs condition

Q_1 = 1,000 cfs
L = 1,000 ft
q_f = 1 cfs/ft
H = 0.5 ft

slope (_ H: 1 V)	2.5	3	4	
α (degrees)	21.80	18.43	14.04	
α (radians)	0.38	0.32	0.24	
C_u	2.1	2.1	2.1	
d_{50} *(ft)*	0.52	0.43	0.33	
selected d_{50} *(ft)*	1	1	1	
V_i	1.59	1.43	1.21	
η	0.45	0.45	0.45	
V_{avg}	0.72	0.64	0.55	
y	1.39	1.55	1.83	
$2*d_{50}$ *(ft)*	2.00	2.00	2.00	$2*d_{50} > y$ *for riprap size to be valid*

2. 1,000 cfs condition

Q_1 = 20,000 cfs
L = 1,000 ft
q_f = 20 cfs/ft
H = 3.7 ft

slope (_ H: 1 V)	2.5	3	4	
α (degrees)	21.80	18.43	14.04	
α (radians)	0.38	0.32	0.24	
C_u	2.1	2	2	
d_{50} *(ft)*	2.49	2.05	1.60	
selected d_{50} *(ft)*	6	6	6.5	
V_i	3.90	3.51	3.09	
η	0.45	0.45	0.45	
V_{avg}	1.76	1.58	1.39	
y	11.39	12.66	14.37	Does not quite meet criteria, but increasing D50 further only widens gap.
$2*d_{50}$ *(ft)*	12.00	12.00	13.00	$2*d_{50} > y$ *for riprap size to be valid*

September 2009
Publication No. FHWA-NHI-09-112

Hydraulic Engineering Circular No. 23

Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance-Third Edition

Volume 2

NATIONAL HIGHWAY INSTITUTE

U.S. Department of Transportation
Federal Highway Administration

Technical Report Documentation Page

1. Report No. FHWA NHI HEC-23	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle

BRIDGE SCOUR AND STREAM INSTABILITY COUNTERMEASURES
Experience, Selection and Design Guidance
Volumes 1 and 2
Third Edition

5. Report Date

September 2009

6. Performing Organization Code

7. Author(s)

P.F. Lagasse, P.E. Clopper, J.E. Pagán-Ortiz, L.W. Zevenbergen, L.A. Arneson, J.D. Schall, L.G. Girard

8. Performing Organization Report No.

9. Performing Organization Name and Address

Ayres Associates
3665 JFK Parkway
Building 2, Suite 200
Fort Collins, Colorado 80525

10. Work Unit No. (TRAIS)

11. Contract or Grant No.

DTFH61-06-D-00010/T-06-001

12. Sponsoring Agency Name and Address

Office of Bridge Technology
FHWA, HIBT-20
1200 New Jersey Ave., SE
Washington, D.C. 20590

National Highway Institute
4600 North Fairfax Dr., Suite 800
Arlington, Virginia 22203

13. Type of Report and Period Covered

14. Sponsoring Agency Code

15. Supplementary Notes

Project Managers: Dr. Larry A. Arneson and Mr. Jorge E. Pagán-Ortiz, FHWA
Technical Assistants: Scott Anderson, Kornel Kerenyi, Joe Krolak, Barry Siel, FHWA; S. Mishra, Ayres Associates; B. Hunt, STV Inc.

16. This document identifies and provides design guidelines for bridge scour and stream instability countermeasures that have been implemented by various State departments of transportation (DOTs) in the United States. Countermeasure experience, selection, and design guidance are consolidated from other FHWA publications in this document to support a comprehensive analysis of scour and stream instability problems and provide a range of solutions to those problems. Selected innovative countermeasure concepts and guidance derived from practice outside the United States are introduced. Management strategies and guidance for developing a Plan of Action for scour critical bridges are outlined, and guidance is provided for scour monitoring using portable and fixed instrumentation.

The results of recently completed National Cooperative Highway Research Program (NCHRP) projects are incorporated in the design guidance, including: countermeasures to protect bridge piers and abutments from scour; riprap design criteria, specifications, and quality control; and environmentally sensitive channel and bank protection measures. This additional material required expanding HEC-23 to two volumes. Volume 1 now contains a complete chapter on riprap design, specifications, and quality control as well as an expanded chapter on biotechnical countermeasures. The guidance on scour monitoring instrumentation has been updated and now includes additional installation case studies. Volume 2 contains 19 detailed design guidelines grouped into six categories, including countermeasures for: (1) stream instability (2) streambank and roadway embankment protection, (3) bridge pier protection, (4) abutment protection, (5) filter design, and (6) special applications.

17. Key Words

stream stability, scour, countermeasures, plan of action, bendway weirs, soil cement, wire enclosed riprap, articulating concrete block systems, concrete armor units, gabion mattresses, grout filled mattresses, grout bags, rock riprap, partially grouted riprap, spurs, guide banks, check dams, revetments, scour monitoring instrumentation

18. Distribution Statement

This document is available to the public through the National Technical Information Service, Springfield, VA 22161 (703) 487-4650

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	376	

Bridge Scour and Stream Instability Countermeasures
Experience, Selection, and Design Guidance
Third Edition

Volume 1

TABLE OF CONTENTS

DESIGN GUIDELINE 5

RIPRAP DESIGN FOR EMBANKMENT OVERTOPPING

5.1 INTRODUCTION

When flow overtops an embankment, spur, or guide bank, locally high velocities and shear stresses will create strong erosion forces, typically at the downstream shoulder and on the embankment slope, that are too great for the soil of the embankment to withstand. Two primary processes of erosion occur during an overtopping event.

When the overtopping flow is submerged, erosion of the embankment typically begins with the downstream shoulder. This condition is often experienced by roadways and bridge approach embankments. Figure 5.1 (Chen and Anderson 1987) shows the progression of this type of failure at times t1, t2, and t3. As the flow accelerates over the embankment, a surging hydraulic jump is formed that causes a nick point between the shoulder and the downstream slope. This nick point will begin to migrate upstream because of the high velocities, and erosion will begin to move downstream. The downstream migration of the erosion is caused by the turbulence associated with the hydraulic jump. This condition would also apply to most river training countermeasures, such as spur and guide banks, under overtopping conditions.

The second general erosion pattern results from the case of free flow. With low tail water, the flow will accelerate down the slope with high velocity and shear stress associated with supercritical flow. Erosion typically initiates near the toe of the embankment, whether or not a hydraulic jump is present. Erosion progresses in the upslope and upstream direction through the embankment. Figure 5.2 (Chen and Anderson 1987) illustrates this progression. This condition would typically apply to earth dams, spillways, or levees protected by revetment riprap.

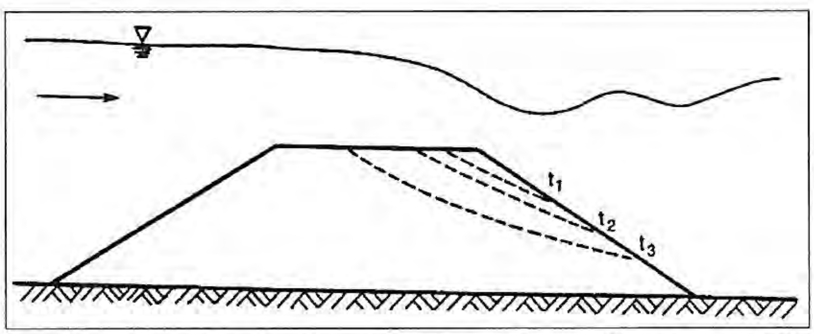


Figure 5.1. Typical embankment erosion pattern with submerged flow.

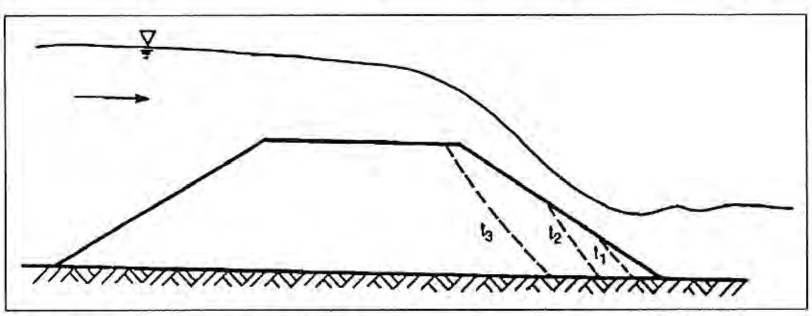


Figure 5.2. Typical embankment erosion pattern with free flow.

Traditionally, riprap has been placed on the downstream slope of embankment dams for erosion protection during heavy rainfall and has commonly been assumed inadequate for protection from overtopping flows. Although prototype verification is limited, several investigators have studied riprap stability on steep embankment slopes when subject to flow. Flow hydraulics on steep embankment slopes cannot be analyzed with standard flow and sediment transport equations. Uniform flow and tractive shear equations do not apply to shallow flow over large roughness elements or highly aerated flow, both of which can occur during overtopping. Riprap design criteria for overtopping protection of embankment dams should prevent stone movement and ensure the riprap layer does not fail. Empirically derived design criteria currently offer the best approach for design (Frizell et al. 1990).

Riprap design to resist overtopping flow is dependent upon the material properties (median size, shape, gradation, porosity, and unit weight), the hydraulic gradient or embankment slope, and the unit discharge. Flume studies were performed to investigate flow through and over rock fill dams, using crushed granite, pebbles, gravel, and cobbles on a range of slopes (Abt et al. 1987, 1988, 1991). Threshold flows where incipient stone movement occurs were defined. The maximum unit discharge that resists stone movement on steep slopes is a function of the mean water depth, the critical velocity at which the stone begins to move, and an aeration factor defined as the ratio of the specific weight of the air-water mixture to the specific weight of the water. A comparison of the various expressions for overtopping flow conditions shows them to be valid for crushed stone with angular shape (Abt and Johnson 1991). Knauss developed a rock stability function based on unit discharge, slope, rock packing, and air concentration for sizing riprap, and determined that aeration of flow increases the critical velocity for which riprap on a steep slope remains stable (Oswalt et al. 1994).

Studies were performed in a near-prototype-size embankment overtopping facility to establish new criteria relating the design of the riprap layer to the interstitial velocity of water flowing through the riprap layer (Mishra 1998). An equation was developed to predict the interstitial velocity of water through the rock layer. A universal formula for designing riprap was derived. This equation was tested for the data obtained in the 1998 study and previous research studies. The universal riprap design equation was found to satisfactorily predict the size of the riprap to be used for a specific unit discharge and a given embankment slope.

5.2 BACKGROUND

Near-prototype flume tests were conducted by CSU (Oswalt et al. 1994) with riprap placed on embankment slopes of 1, 2, 8, 10, and 20% and subjected to overtopping flows until failure. Failure was defined by exposure of the underlying sand and gravel bedding. Based on the results of five tests, rounded-shape riprap was found to fail at a unit discharge about 40% less that that of angular stones of the same median size, demonstrating the importance of stone shape on riprap layer stability. Angular stones tend to wedge or interlock and require fewer fines to fill voids, compared to similarly graded round stones. Rounded stones are much more likely to slide or roll, especially on the steeper slopes. Riprap specifications normally require angular shaped stone.

Channelization was observed to occur between the threshold and collapsing stages of the overtopping flow. Channels form in the riprap layer as the smaller stones are washed out, producing flow concentrations and increasing the localized unit discharge. Studies at Colorado State University (CSU) suggest flow concentrations of three times the normal unit discharge are possible. The average point of incipient channel formation was identified at about 88% of the unit discharge at failure.

Wittler and Abt (1990) investigated the influence of material gradation and the stability of the riprap layer with overtopping flow. In general, uniformly graded riprap displays a greater stability for overtopping flows but fails suddenly, while well-graded riprap resists sudden failure as voids are filled with smaller material from upstream; this process is referred to as "healing." Additional studies at CSU from 1994-1997 provided more details on the failure mechanism (Mishra 1998). Again, failure of the riprap slope was defined as removal or dislodgment of enough material to expose the bedding material. Failure of the riprap layer occurred with the measured water depth still within the thickness of the rock layer. A layer of highly aerated water was flowing over the surface of the riprap, but this surface flow was only a small portion of the total flow (see Figure 5.10).

5.3 LABORATORY STUDIES

Through the cooperative agreement signed in 1991, the U.S. Bureau of Reclamation (USBR) and Colorado State University (CSU) built a near-prototype size embankment overtopping research facility with a 50% slope (1V:2H). Riprap (angular) tests were conducted in the summers of 1994, 1995, and 1997 on this facility (Mishra 1998).

The first two riprap test sections covered the full width of the chute and extended 60 ft (18.29 m) down the slope from the crest. The first test (1994) consisted of a 0.67-foot (203-mm) thick gravel bedding material with a 2-foot (0.61-m) overlay of large riprap with a d_{50} of 1.27 ft (386 mm) (Figure 5.3). The second test (1995) utilized the first test bed with a second layer of approximately 2 ft (0.61 m) thick riprap with d_{50} of 2.15 ft (655 mm). The schematic diagram for this set up is presented in Figure 5.5.

The third test (1997) covered the full width of the chute and extended 100 ft (30.48 m) from the crest down the slope to the toe of the facility. A 0.67-foot (203-mm) thick gravel bedding material with a d_{50} of 0.16 ft (48 mm) was overlaid with a main riprap layer of thickness 1.75 ft (533 mm) with a d_{50} of 0.89 ft (271 mm). A berm was built at the bottom of the flume to simulate toe treatment at the base of the embankment. The configuration of the test setup in 1997 is given in Figure 5.4. The schematic diagram for the 1997 setup is illustrated in Figure 5.6.

For all the tests, a gabion composed of the same rocks used on the slope, was placed at the crest of the embankment. This was done to provide a smooth transition of water from the head box to the embankment and to prevent premature failure of the riprap at the transition between the concrete approach at the crest of the embankment and the concrete chute. The gabion covered the entire width of the flume and extended about 2.46 ft (0.75 m) down the flume from the crest. The top surface of the gabion was horizontal.

The test series provided the opportunity to gather important data regarding flow through large size riprap. The visual observations provided information on aeration, interstitial flow, stone movement, and the failure mechanism on the slope. Data was collected on discharge flowing down the chute through the riprap, the head box depth for overtopping heads, manometer readings for depth of flow down the chute and the pressure heads, and electronic recording of electrical conductivity versus time to determine interstitial velocities.

Figure 5.3. Test set up for 1994, d_{50} = 1.27 ft (386 mm).

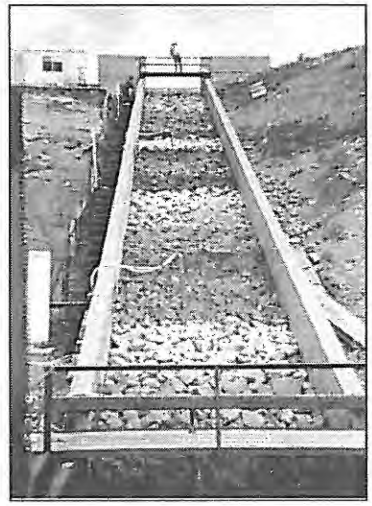


Figure 5.4. Test set up for 1997, d_{50} = 0.89 ft (271 mm).

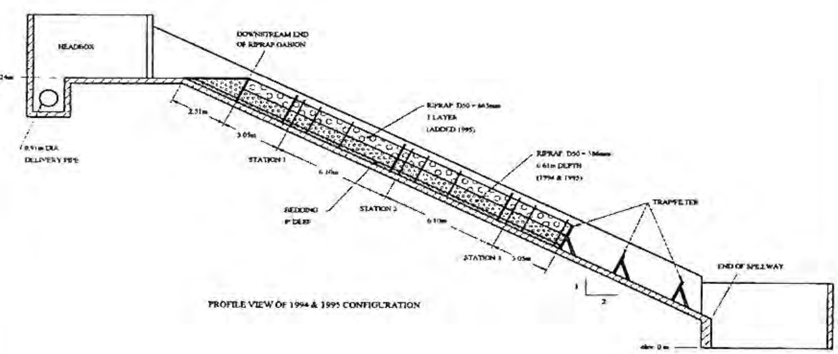


Figure 5.5. Riprap configuration in 1994 and 1995.

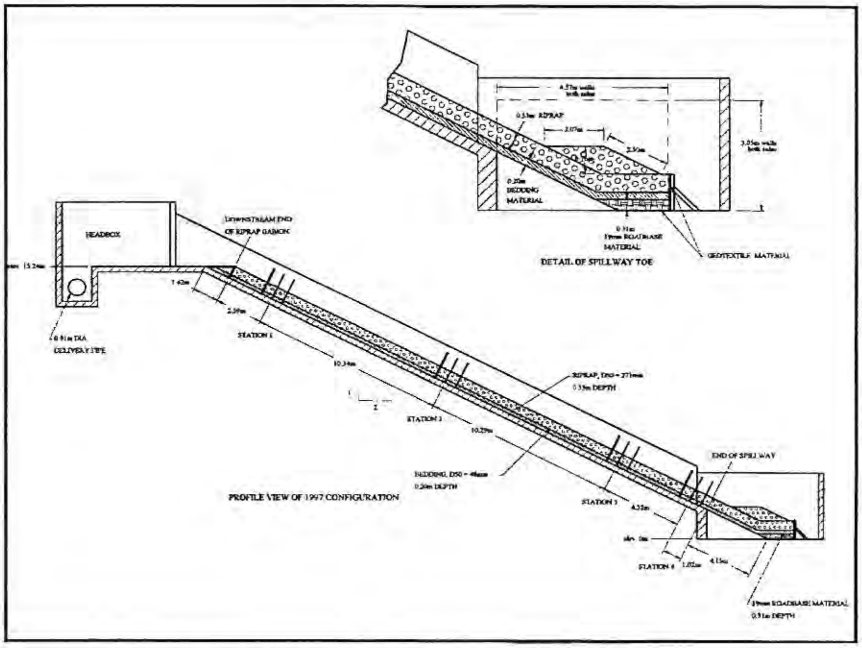


Figure 5.6. Riprap Configuration in 1997.

5.4 RIPRAP FAILURE ON EMBANKMENT SLOPES

Prior to failure of the riprap slope, many individual stones moved or readjusted locations throughout the test period. Movement of these stones is referred to as incipient motion. This occurs when the displacing and overturning moments exceed the resisting moments. The force in the resisting moment is given by the component of the weight perpendicular to the embankment and interlocking between stones in the matrix. The overturning forces are the drag (or the jet impact on a stone), the lift, buoyancy, and to a lesser degree, the component of the weight parallel to the embankment depending on the point(s) of contact with other stones. Even though buoyancy plays an important role in the removal of rocks, the hydrodynamic forces have the major role in producing failure of the protective layer. This observation is supported by the depth measurements, which revealed that the stones on the surface were not entirely submerged. It was also concluded that on steep embankments, riprap failure on the slope is more critical than the failure at the toe.

Failure of the riprap slope was defined as removal or dislodgement of enough material to expose the bedding material. Failure of the riprap layer occurred with the measured water depth still within the thickness of the rock layer. A layer of highly aerated water was flowing over the surface of the riprap, but this surface flow was only a small portion of the total flow and was not measurable by piezometers. Riprap failures are illustrated in Figures 5.7, 5.8, and 5.9 and failure characteristics are given in Table 5.1.

Table 5.1. Riprap Failure Characteristics.			
Year	d_{50} ft (mm)	Coefficient of Uniformity, C_u (d_{60}/d_{10})	Failure Discharge $ft^3/s/ft$ ($m^3/s/m$)
1994	1.27 (386)	1.90	2.4 (0.223)
1995	2.15 (655)	1.55	10 (0.929)
1997	0.89 (271)	1.81	2.2 (0.204)

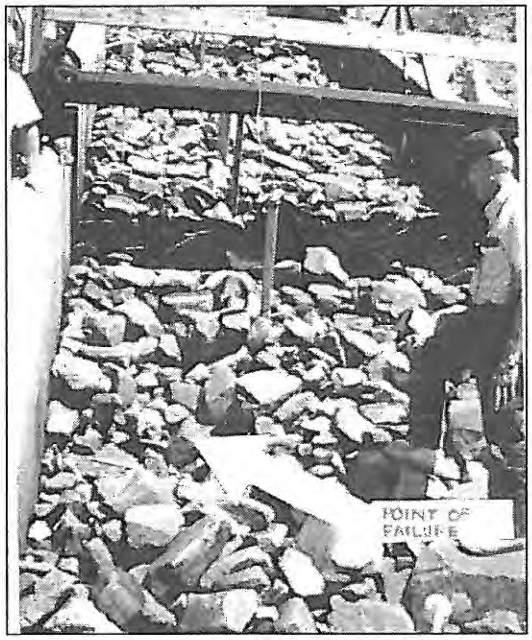


Figure 5.7. Riprap failure in 1994 tests (d_{50} = 1.27 ft (386 mm))

Figure 5.8. Failure of riprap in 1995 tests (d_{50} = 2.15 ft (655 mm))

Figure 5.9. Riprap failure in 1997 tests (d_{50} = 0.89 ft (271 mm)).

5.5 DESIGN OF RIPRAP FOR OVERTOPPING FLOW

5.5.1 Sizing the Riprap

When flow overtops an embankment, spur, or guide bank, locally high velocities occur at the downstream shoulder of the structure. When tailwater is low relative to the crest of the structure, the flow will continue to accelerate along the downstream slope. Guidance for riprap stability under these conditions was developed from the laboratory testing described in Section 5.3 (Mishra 1998). For slopes steeper than 1V:4H, the method requires that all the flow is contained within the thickness of the riprap layer (interstitial flow). For milder slopes, a portion of the total discharge can be carried over the top of the riprap layer. The three equations necessary to assess the stability of rock riprap in overtopping flow are given below. The design procedure is illustrated by examples in Sections 5.5.2 and 5.5.3.

$$V_i = 2.48\sqrt{gd_{50}}\left(\frac{S^{0.58}}{C_u^{2.22}}\right) \tag{5.1}$$

where:
- V_i = Interstitial velocity, ft/s (m/s)
- g = Acceleration due to gravity, 32.2 ft/s^2 (9.81 m/s^2)
- d_{50} = Particle size for which 50% is finer by weight, ft (m)
- C_u = Coefficient of uniformity of the riprap, d_{60}/d_{10}
- S = Slope of the embankment, ft/ft (m/m)

$$d_{50} = \frac{K_u q_f^{0.52}}{C_u^{0.25} S^{0.75}}\left(\frac{\sin\alpha}{(S_g\cos\alpha - 1)(\cos\alpha\tan\phi - \sin\alpha)}\right)^{1.11} \tag{5.2}$$

where:

d_{50}	=	Particle size for which 50% is finer by weight, ft (m)
K_u	=	0.525 for English units 0.55 for SI units
q_f	=	Unit discharge at failure, $ft^3/s/ft$ ($m^3/s/m$)
C_u	=	Coefficient of uniformity of the riprap, d_{60}/d_{10}
S	=	Slope of the embankment, ft/ft (m/m)
S_g	=	Specific gravity of the riprap
α	=	Slope of the embankment, degrees
φ	=	Angle of repose of the riprap, degrees

When the embankment slope is less than 1V:4H (25%), the allowable depth of flow (h) over the riprap is given by:

$$h = \frac{0.06(S_g - 1)d_{50}\tan\phi}{0.97(S)} \tag{5.3}$$

5.5.2 Example Application for Slopes Milder Than 1V:4H (25%)

Riprap is to be designed to protect a 1V:5H slope from overtopping flow. The riprap has a specific gravity (S_g) of 2.65, uniformity coefficient (C_u) of 2.1, porosity η of 0.45 and an angle of repose ϕ of 42°. The following data are provided for the design.

Variable	English Units		SI Units	
	Units	Value	Units	Value
Total discharge (Q)	cfs	2000	m^3/s	56.63
Embankment overtopping length (L)	ft	1000	m	304.8
Unit discharge (q_f)	cfs/ft	2.0	m^2/s	0.186
Weir flow coefficient (C)	$ft^{0.5}/s$	2.84	$m^{0.5}/s$	1.57
Riprap sizing equation coefficient (K_u)	$s^{0.52}/ft^{0.04}$	0.525	$s^{0.52}/m^{0.04}$	0.55
Manning-Strickler coefficient		0.034		0.0414
Slope (S)	ft/ft	0.2	m/m	0.2
Slope angle (α)	degrees	11.3	degrees	11.3

Step 1: Determine the overtopping depth using the broad-crested weir equation:

$$Q = CLH^{1.5}$$

$$H = (Q/CL)^{2/3} = [2000/(2.84x1000)]^{2/3} = 0.79 \text{ ft } (0.24 \text{ m})$$

Step 2: Compute the smallest possible median rock size (d_{50}) using Equation 5.2:

$$d_{50} = \frac{K_u q_f^{0.52}}{C_u^{0.25} S^{0.75}} \left(\frac{\sin\alpha}{(S_g \cos\alpha - 1)(\cos\alpha \tan\phi - \sin\alpha)} \right)^{1.11}$$

$$= \frac{0.525(2.0)^{0.52}}{(2.1)^{0.25}(0.2)^{0.75}} \left(\frac{\sin(11.3^\circ)}{[2.65\cos(11.3^\circ) - 1]\,[\cos(11.3^\circ)\tan(42^\circ) - \sin(11.3^\circ)]} \right)^{1.11}$$

$$= 0.31 \text{ ft} = 3.7 \text{ inches } (0.094 \text{ m})$$

Note: Use next larger size class (see Volume 1, Chapter 5).

Step 3: Select Class I riprap from Table 4.1 of Design Guideline 4: d_{50} = 6 inches (0.15 m)

Step 4: Compute the interstitial velocity and the average velocity using Equation 5.1:

$$V_i = 2.48\sqrt{gd_{50}}\,\frac{S^{0.58}}{C_u^{2.22}} = 2.48\sqrt{32.2(0.5)}\,\frac{(0.2)^{0.58}}{(2.1)^{2.22}}$$
$$= 0.75\ \text{ft/s}\ (0.228\ \text{m/s})$$

From V_i, find the average velocity V_{avg}

$V_{ave} = \eta\ V_i = 0.45(0.75) = 0.34$ ft/s (0.103 m/s)

where: η is the porosity of the rock.

Step 5: Compute the average flow depth (y) as if all the flow is contained within the thickness (t) of the riprap layer (i.e., t = y):

$y = q_f/V_{avg} = 2.0/0.34 = 5.9$ ft (1.81 m)

NOTE: If the average depth is less than $2d_{50}$ then the design is complete with a riprap thickness of $2d_{50}$. If the depth is greater than $2d_{50}$ and the slope is greater than 0.25, go to Step 11. Otherwise, go to Step 6.

5.9 ft > $2d_{50}$ (1.0 ft) and S (0.2) < 0.25, so go to step 6.

Step 6: Find the allowable flow depth over the riprap using Equation 5.3:

$$h = \frac{0.06(S_g - 1)d_{50}\tan\phi}{0.97(S)} = \frac{0.06(2.65-1)(0.5)\tan 42^\circ}{0.97(0.2)}$$
$$= 0.23\ \text{ft}\ (0.069\ \text{m})$$

Step 7: Calculate the Manning roughness coefficient, n

$$n = 0.034(d_{50})^{1/6} = 0.034(0.5)^{1/6} = 0.030$$

Step 8: Calculate the unit discharge, q_1, which can flow over the riprap using Manning's equation:

$$q_1 = \frac{1.486}{n}y^{5/3}S^{1/2} = \frac{1.486}{0.03}(0.23)^{5/3}(0.2)^{1/2}$$
$$= 1.91\ \text{ft}^3/\text{s}/\text{ft} = 0.173\ \text{m}^3/\text{s}/\text{m}$$

Step 9: Calculate the required interstitial flow, q_2, through the riprap and the flow provided by a riprap thicknesses of $2d_{50}$.

$q_2 = q_f - q_1 = 2.0 - 1.91 = 0.09$ ft^3/s/ft (0.013 m^3/s/m)

$q = 2d_{50}(V_{avg}) = 2(0.5)(0.34) = 0.34$ ft^3/s/ft (0.031 m^3/s/m)

NOTE: If the flow (q) provided by a $2d_{50}$ thickness is greater than or equal to the required flow (q_2), the design is complete with a thickness of $2d_{50}$. If the flow provided by $2d_{50}$ is less than the required flow, proceed to Step 10.

$$q\ (0.34\ ft^3/s/ft) > q_2\ (0.09\ ft^3/s/ft)$$

Therefore, the design is complete using a thickness of $2d_{50}$ and a riprap d_{50} of 6 inches.

Step 10: (not needed for this example). Calculate the flow provided by a $4d_{50}$ thickness of riprap. If the flow provided is greater than the required flow, the design is complete with a thickness of $4d_{50}$ (or an appropriate intermediate thickness). If the flow provided by a $4d_{50}$ thickness is less than the required flow, proceed to Step 11.

Step 11: (not needed for this example). Increase the riprap size to the next gradation class and return to Step 4.

5.5.3 Example Application for Slopes Steeper Than 1V:4H (25%)

Using the same data as the previous example, design riprap for a 1V:2H slope (50%). Because the slope is steeper than 1V:4H, the riprap is designed such that all the flow is through the riprap (interstitial flow).

Variable	English Units		SI Units	
	Units	Value	Units	Value
Total discharge (Q)	cfs	2000	m^3/s	56.63
Embankment overtopping length (L)	ft	1000	m	304.8
Unit discharge (q_f)	cfs/ft	2.0	m^2/s	0.186
Weir flow coefficient (C)	$ft^{0.5}/s$	2.84	$m^{0.5}/s$	1.57
Riprap sizing equation coefficient (K_u)	$s^{0.52}/ft^{0.04}$	0.525	$s^{0.52}/m^{0.04}$	0.55
Manning-Strickler coefficient		0.034		0.0414
Slope (S)	ft/ft	0.5	m/m	0.5
Slope angle (α)	degrees	26.6	degrees	26.6

Step 1: Determine the overtopping depth using the broad-created weir equation:

$$Q = CLH^{1.5}$$

$$H = (Q/CL)^{2/3} = [2000/(2.84x1000)]^{2/3} = 0.79\ ft\ (0.24\ m)$$

Step 2: Compute the smallest possible median rock size (d_{50}):

$$d_{50} = \frac{K_u q_f^{0.52}}{C_u^{0.25} S^{0.75}} \left(\frac{\sin\alpha}{(S_g \cos\alpha - 1)(\cos\alpha \tan\phi - \sin\alpha)} \right)^{1.11}$$

$$= \frac{0.525(2.0)^{0.52}}{(2.1)^{0.25}(0.5)^{0.75}} \left(\frac{\sin(26.6^\circ)}{[2.65\cos(26.6^\circ) - 1][\cos(26.6^\circ)\tan(42^\circ) - \sin(26.6^\circ)]} \right)^{1.11}$$

$$= 0.96\ ft = 11.5\ inches\ (0.29\ m)$$

Step 3: Select Class III riprap from Table 4.1 of Design Guideline 4: d_{50} = 12 inches (0.15 m).

Step 4: Compute the interstitial velocity and the average velocity:

$$V_i = 2.48\sqrt{gd_{50}}\frac{S^{0.58}}{C_u^{2.22}} = 2.48\sqrt{32.2(1.0)}\frac{(0.5)^{0.58}}{(2.1)^{2.22}}$$
$$= 1.81 \text{ ft/s } (0.548 \text{ m/s})$$

$V_{ave} = \eta \ V_i \ = 0.45(1.81) = 0.81$ ft/s (0.247 m/s)

Step 5: Compute the average flow depth (y) as if all the flow is contained within the thickness (t) of the riprap layer (i.e., t = y):

$y = q_f/V_{avg} = 2.0/0.81 = 2.5$ ft (0.75 m)

NOTE: If the average depth is less than $2d_{50}$ then the design is complete with a riprap thickness of $2d_{50}$. If the depth is greater than $2d_{50}$ and the slope is greater than 0.25, go to step 11. Otherwise, go to Step 6.

2.5 ft > $2d_{50}$ (2.0 ft) and S (0.5) > 0.25, so go to Step 11.

Step 11: Increase the riprap size to the next gradation class and return to Step 4.

Step 12: Select Class IV riprap with d_{50} of 15 inches from Table 4.1 (Design Guideline 4) and return to Step 4.

Step 4 (trial 2): Compute the interstitial velocity and the average velocity:

$$V_i = 2.48\sqrt{gd_{50}}\frac{S^{0.58}}{C_u^{2.22}} = 2.48\sqrt{32.2(1.25)}\frac{(0.5)^{0.58}}{(2.1)^{2.22}}$$
$$= 2.03 \text{ ft/s } (0.617 \text{ m/s})$$

$V_{ave} = \eta \ V_i \ = 0.45(2.03) = 0.91$ ft/s (0.278 m/s)

Step 5 (trial 2): Compute the average flow depth (y) as if all the flow is contained within the thickness (t) of the riprap layer (i.e., t = y):

$y = q_f/V_{avg} = 2.0/0.91 = 2.2$ ft (0.67 m)

NOTE: If the average depth is less than $2d_{50}$ then the design is complete with a riprap thickness of $2d_{50}$. If the depth is greater than $2d_{50}$ and the slope is greater than 0.25, go to Step 11. Otherwise, go to Step 6.

2.2 ft < $2d_{50}$ (2.5 ft), so design is complete with d_{50} = 15 inches and a riprap thickness of 2.5 feet. This check ensures that all the flow is contained within the thickness of the riprap layer (interstitial flow).

5.6 FILTER REQUIREMENTS

The importance of the filter component of any embankment riprap installation should not be underestimated. Geotextile filters and granular filters may be used in conjunction with riprap embankment protection. When using a granular stone filter, the layer should have a minimum thickness of 4 times the d_{50} of the filter stone or 6 inches, whichever is greater.

The filter must retain the coarser particles of the subgrade while remaining permeable enough to allow infiltration and exfiltration to occur freely. It is not necessary to retain all the particle sizes in the subgrade; in fact, it is beneficial to allow the smaller particles to pass through the filter, leaving a coarser substrate behind. Detailed aspects of filter design are presented in Design Guideline 16 of this document.

5.8 REFERENCES

Abt, S.R. and Johnson, T.L., 1991, "Riprap Design for Overtopping Flow," ASCE Journal of Hydraulic Engineering, Vol. 117, No. 8, pp. 959-972.

Abt, S.R., Khattak, M.S., Nelson, J.D., Ruff, J.F., Shaikh, A., Wittler, R.J., Lee, D.W., and Hinkle, N.E., 1987, "Development of Riprap Design Criteria by Riprap Testing in Flumes: Phase I," NUREG/CR-4651, U.S. Nuclear Regulatory Commission, Vol. 1, 48-53.

Abt, S.R., Wittler, R.J., Ruff, J.F., Lagrone, D.L., Nelson, J.D., Hinkle, N.E., Lee, D.W., and 1988, "Development of Riprap Design Criteria by Riprap Testing in Flumes: Phase II," NUREG/CR-4651, U.S. Nuclear Regulatory Commission, Vol. 2, 57-65.

Abt. S.R., Ruff, J.F., and Wittler, R.J., 1991, "Estimating Flow Through Riprap," ASCE Journal of Hydraulics, Vol. 5, 670-675.

Chen, Y.H. and Anderson, B.A., 1987, "Development of a Methodology for Estimating Embankment Damage Due to Flood Overtopping," FHWA Report No. FHWA-RD-86/126.

Frizell, K.H., Mefford, B.W., Dodge, R.A., and Vermeyen, T.B., 1990, "Protecting Embankment Dams Subject to Overtopping During Major Flood Events," Proceedings American State Dam Safety Officials Conference, New Orleans, LA.

Oswalt, N.R., Buck, L.E., Hepler, T.E., and Jackson, H.E., 1994, "Alternatives for Overtopping Protection of Dams," ASCE, Task Committee on Overtopping Protection of the Hydraulics Division, New York, NY, pp 136.

Mishra, S.K., 1998, "Riprap Design for Overtopped Embankments," Ph.D. Dissertation, Department of Civil Engineering, Colorado State University, Fort Collins, CO, 140 pp.

Robinson, K.M., Rice, C.E., and Kadavy, K.C., 1995, "Stability of Rock Chutes," Proceedings, Water Resources Engineering, ASCE, San Antonio, TX, Vol. 2, 1476-1480.

Whittler, R.J. and Abt, S.R., 1990, "The Influence of Uniformity on Riprap Stability," Proceedings, Hydraulic Engineering Vol. 1, of the 1990 ASCE National Conference, San Diego, CA, July 30-August 3, pp. 251-265.

Attachment E
CD of Attachments and References

DENVER
2490 W. 26th Avenue Suite 100A
Denver, Colorado 80211
Phone: 303.480.1700
Fax: 303.480.1020

GLENWOOD SPRINGS
818 Colorado Avenue
P.O.Box 219
Glenwood Springs, Colorado 81602
Phone: 970.945.7755
Fax: 970.945.9210

DURANGO
1666 N. Main Avenue Suite C
Durango, Colorado 81301
Phone: 970.259.7411
Fax: 970.259.8758

www.wrightwater.com

Wright Water Engineers, Inc.