Dam and Levee Owners' Perspective:
Issues and Engineering Needs
US Army Corps of Engineers

Scott Shewbridge, PhD, PE, GE (email Scott.E.Shewbridge@usace.army.mil)
US Army Corps of Engineers, Risk Management Center

International Workshop on Overflowing Erosion of Dams and Dikes

Aussois, France

December 11, 2017

US Army Corps of Engineers®

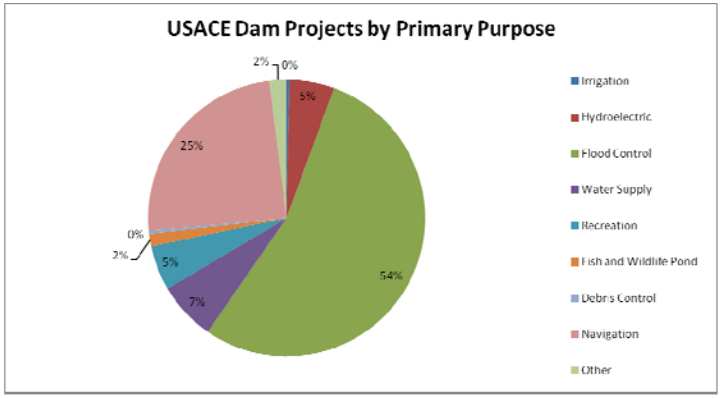
Presentation Topics

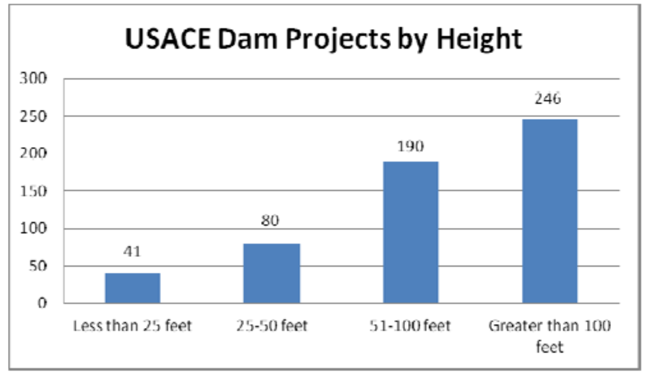
- US Army Corps of Engineers Inventory and Performance Challenges
- Flood Risk Management Erosion Issues
  - Risk Analysis – Hazard, Performance and Consequences
  - Potential Failure Modes Analysis and Event Trees
  - Consequences
  - Risk-Informed Design Progression
- Erosion Engineering Needs
  - Models
  - Parameters
    - Critical Shear Stress
    - Erosion Coefficient
    - Wave Overtopping Erosion Thresholds and Rates
- Discussion/Questions

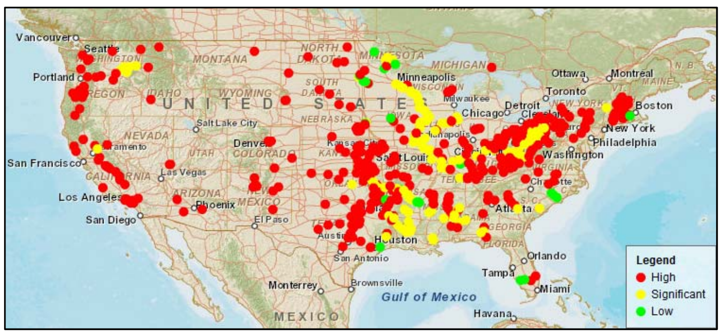
Presentation Topics

- US Army Corps of Engineers Inventory and Performance Challenges
- Flood Risk Management Erosion Issues
  - Risk Analysis – Hazard, Performance and Consequences
  - Potential Failure Modes Analysis and Event Trees
  - Consequences
  - Risk-Informed Design Progression
- Erosion Engineering Needs
  - Models
  - Parameters
    - Critical Shear Stress
    - Erosion Coefficient
    - Wave Overtopping Erosion Thresholds and Rates
- Discussion/Questions

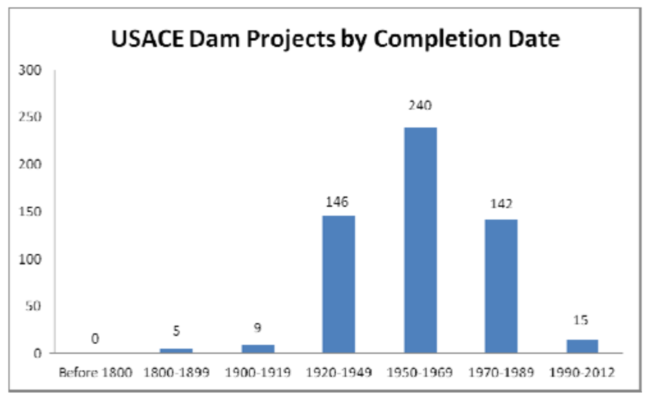
USACE Inventory of Dams







650+ Dams
72% “High Hazard”



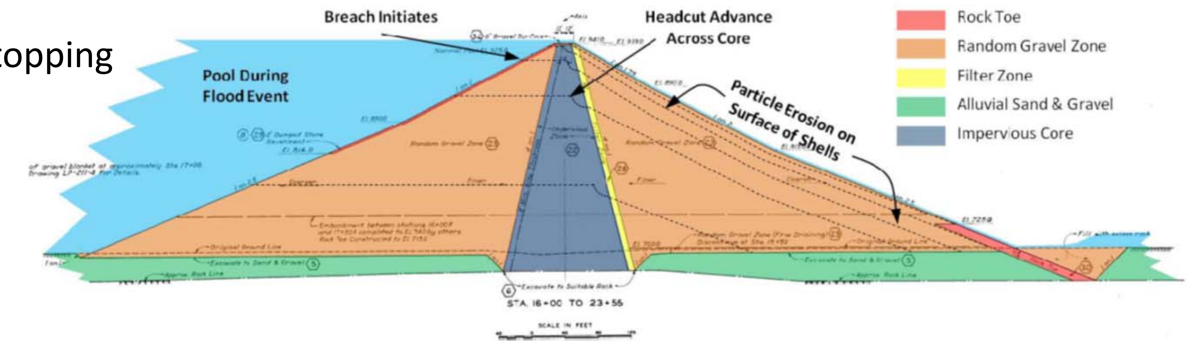
Average Age = 55 years old

Dam Internal, Overtopping and Spillway Erosion Examples

Teton Dam

Internal Erosion

Overtopping



Spillway Erosion

USACE Canyon Dam Spillway

Levees and Floodwalls

- About 15,000 miles in the National Levee Database with USACE Nexus (typically USACE Designed and Built, Local Sponsor Operated)
- USACE Operated limited generally to lower Mississippi Valley below Memphis District
- Probably 50,000 to 100,000+ miles in the nation per NLSC reports

Levee Erosion Examples

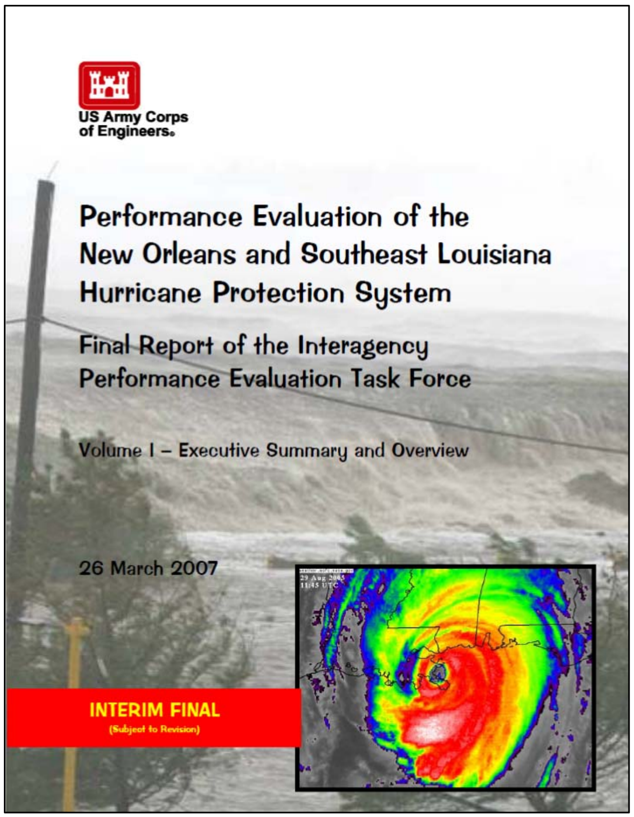


Figure 13-14. IHNC East, Approximate B/L Sta 101+00 (from drawing file H-2-24111, plate IV-23, DM2 Supp 8 IHNC Remaining Levees), North of Chef Menteur Hwy Bridge. Top of I-wall is elev 14.75, bottom of concrete is elev 7, and levee crown is elev 9. Nearest B/L boring is Sta 96+00 (No. 9EU), 500 feet distant. Approximate storm surge impact was a 2.5-ft water crest cascading over the 6-ft concrete wall. Note that the scour was deeper than the concrete base, indicating that the structural backfill and the original levee material eroded

Katrina Flood Wall and Embankment Overtopping Erosion

Figure 22. Example of breach along IHNC (east side) from overtopping and scour (top) and scour behind adjacent section that did not f fail (bottom).

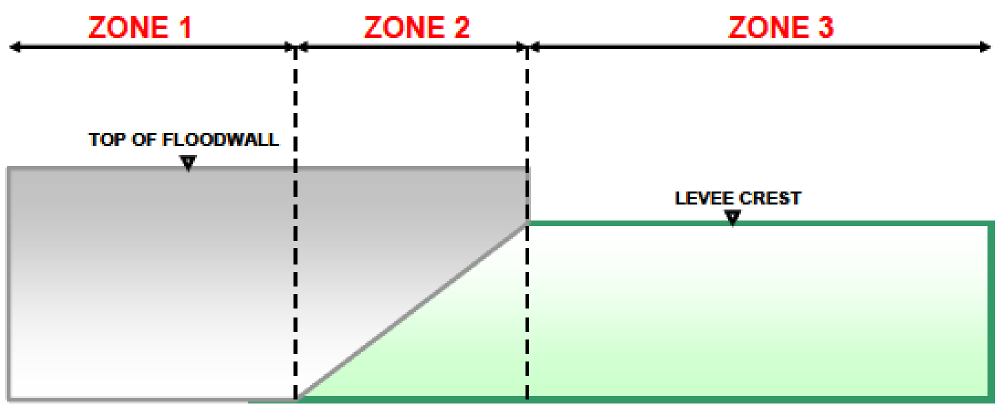
Initial Stage A: Erosion due to overtopping on the Citrus Back Levee

Stage B: Headcut erosion along the IHNC

Stage C: Crown scour along the MRGO levee in St. Bernard Parish

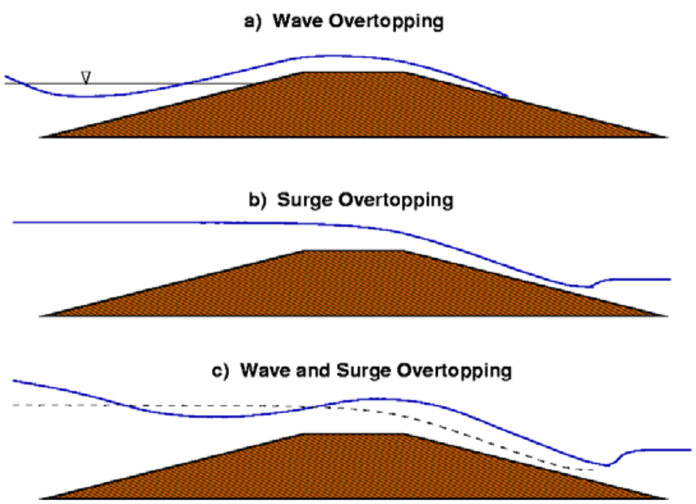
Stage D: Overtopping erosion at Bayou Dupre in St. Bernard Parish

Transition Area Armoring



- Zone 1 is the floodwall-only section;
- Zone 2 is the sloping section of levee that overlaps with the floodwall
- Zone 3 is the levee-only section

Transient Wave Loading



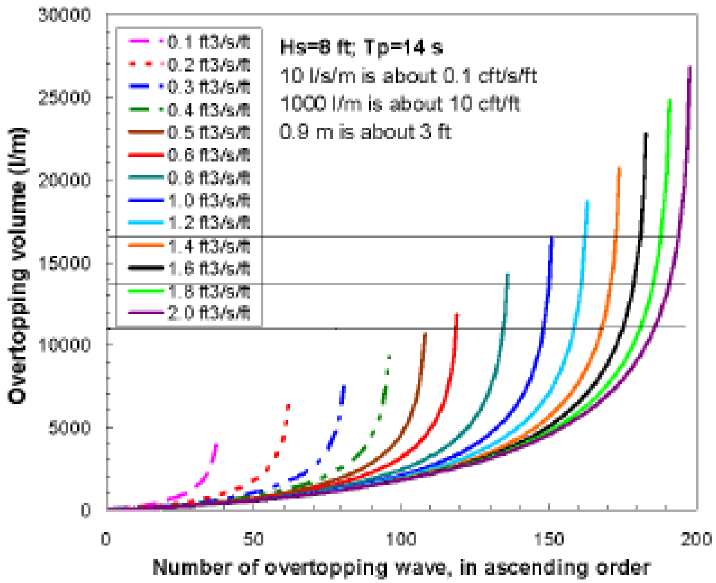


Figure 2.3. Required distribution of overtopping volumes for H_{m0} = 8 ft with T_p = 14 s.

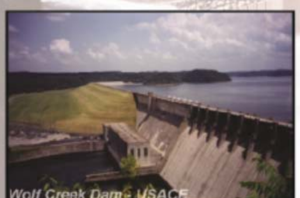
2011 Missouri River Flood - Riverside Scour

Presentation Topics

- US Army Corps of Engineers Inventory and Performance Challenges
- Flood Risk Management Erosion Issues
  - Risk Analysis – Hazard, Performance and Consequences
  - Potential Failure Modes Analysis and Event Trees
  - Consequences
  - Risk-Informed Design Progression
- Erosion Engineering Needs
  - Models
  - Parameters
    - Critical Shear Stress
    - Erosion Coefficient
    - Wave Overtopping Erosion Thresholds and Rates
- Discussion/Questions

Best Practices in Dam and Levee Safety Risk Analysis

Dallas Floodway - USACE



Wolf Creek Dam - USACE

background: Hoover Dam - BOR

U.S. Department of the Interior
Bureau of Reclamation

U.S. Army Corps of Engineers
BUILDING STRONG®

Version 4.0
July 2015

Best Practices in Dam And Levee Safety Risk Analysis

A Joint Publication by
U.S. Department of the Interior, Bureau of Reclamation, and
U.S. Army Corps of Engineers

With Contributions By:

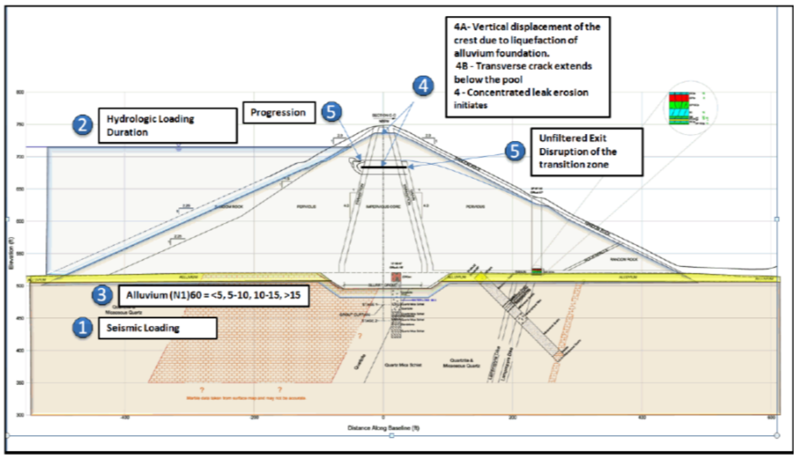
Bureau of Reclamation

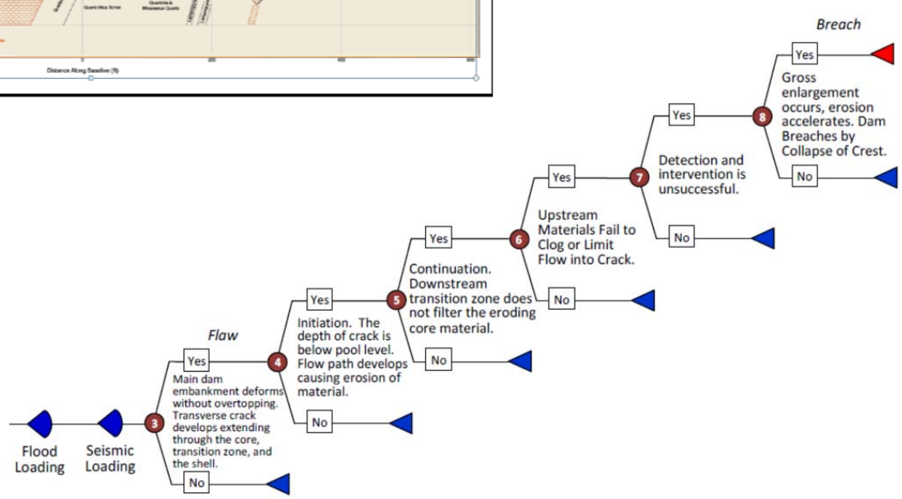
William R. Fiedler, P.E.
Civil Engineer
John England, PhD, P.H., P.E.
Hydraulic Engineer
William O. Engemoen, P.E.
Geotechnical Engineer
Daniel Levish, PhD
Seismologist
Steve Dominic, P.E.
Structural Engineer
Phoebe Percell, P.E.
Structural Engineer
Daniel Osmun, P.E.
Geotechnical Engineer
Larry K. Nuss, P.E.
Structural Engineer
John Trojanowski, P.E.
Civil Engineer
David R. Gillette, PhD, P.E.
Geotechnical Engineer
Larry W. Anderson, P.G.
Geologist
Pencheng Liu
Seismologist
Dom Galic, PhD, P.E.
Geotechnical Engineer
Bruce Feinberg, P.E.
Hydraulic Engineer

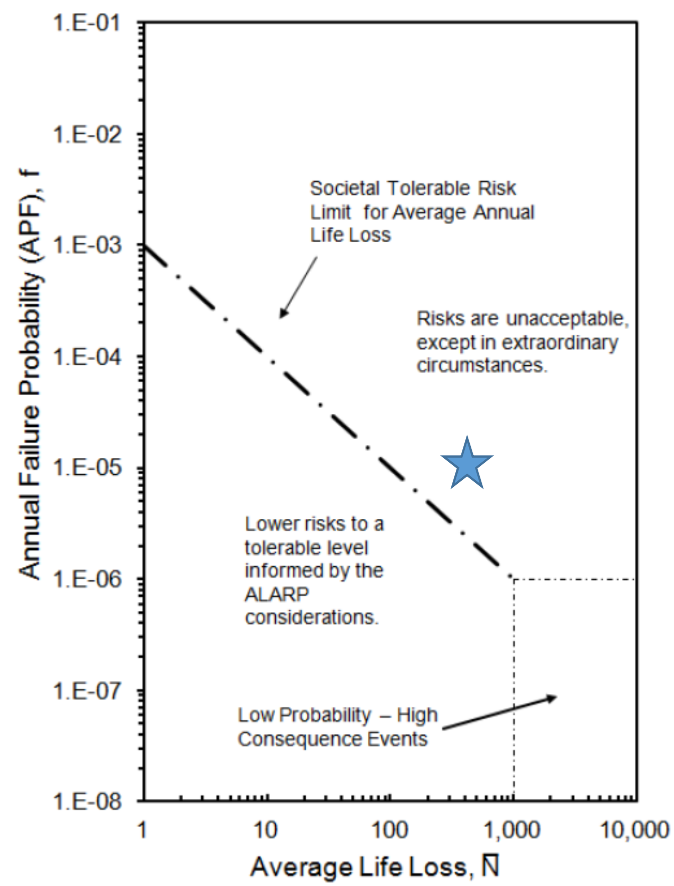
Corps of Engineers

Gregg A. Scott, P.E.
Civil Engineer
Karl M. Dise, P.E.
Geotechnical Engineer
Nathan J. Snorteland, P.E.
Geotechnical Engineer
Rick W. Schultz, P.E.
Mechanical Engineer
Timothy M. O'Leary, P.E.
Geotechnical Engineer
Charles G. Redlinger, P.E.
Geotechnical Engineer
Jeffrey T. McClenathan, P.E.
Hydraulic Engineer
David A. Margo, P.E.
Hydraulic Engineer
David M. Schaaf, P.E.
Structural Engineer
Jeffrey A. Schaefer, PhD, P.G., P.E.
Geotechnical Engineer
Peter T. Shaffner, P.G.
Geologist
Jason T. Needham, P.E.
Hydraulic Engineer
Kent G. Walker, P.E.
Hydraulic Engineer
Thomas Terry, P.G.
Geologist
Scott E. Shewbridge, PhD, P.E.
Geotechnical Engineer

Failure Mode, Event Tree, Risk Plot

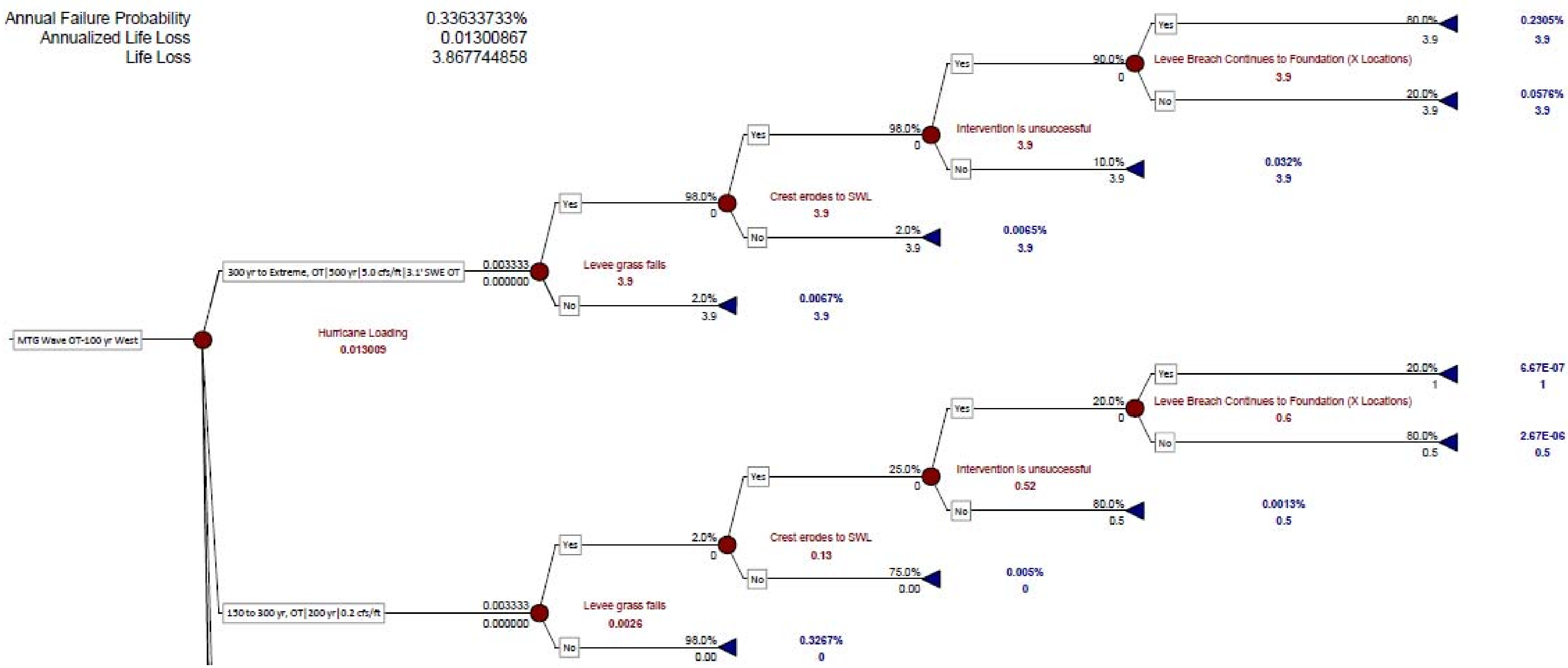




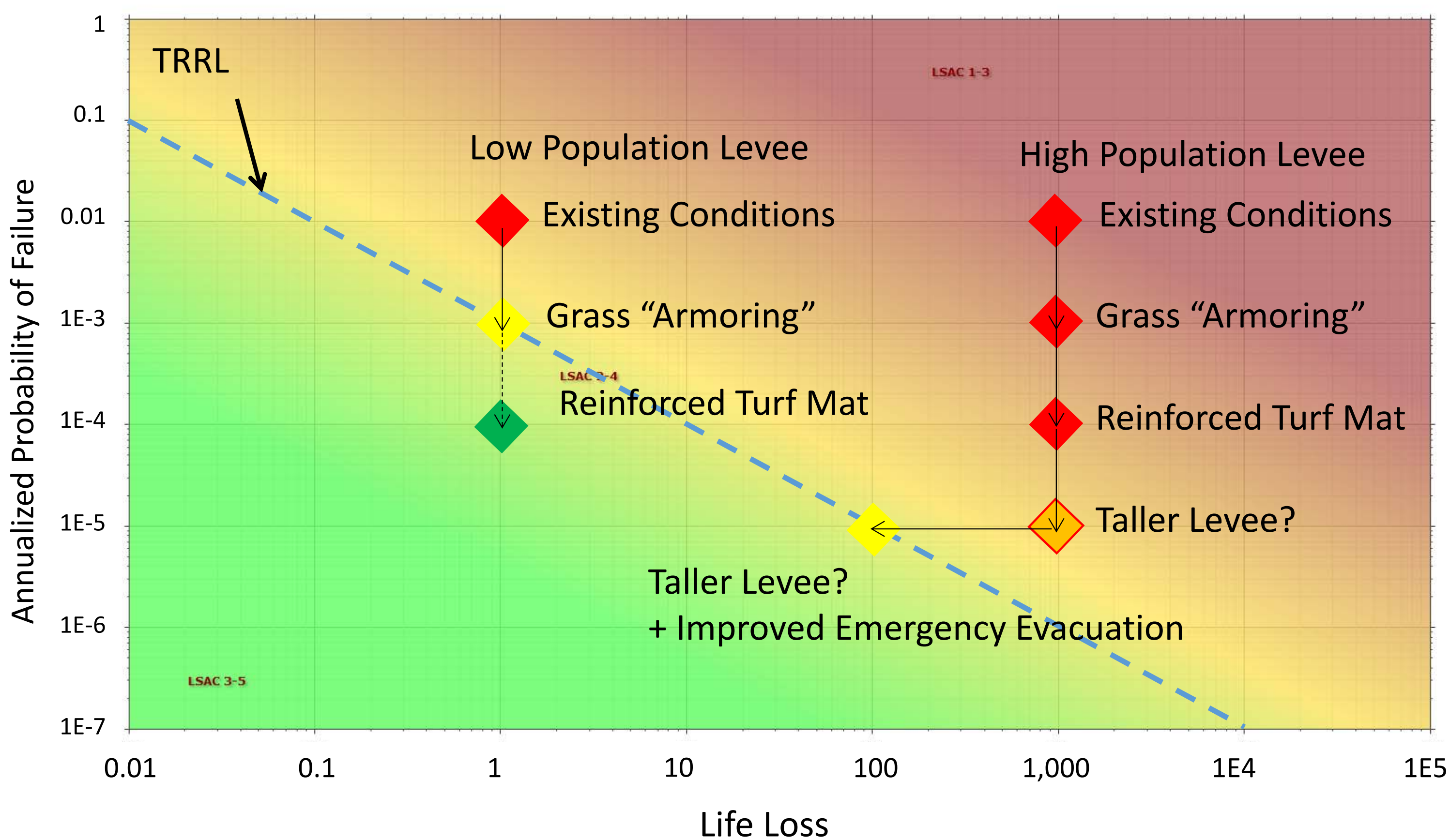


Hurricane Wind Wave Overtopping Erosion Event Tree

100 Yr Levee - HSDRRS Criteria
West Levee, Reach A South Crest elev = 17.5 ft



Risk-Informed Design Progression
Alternatives Analysis



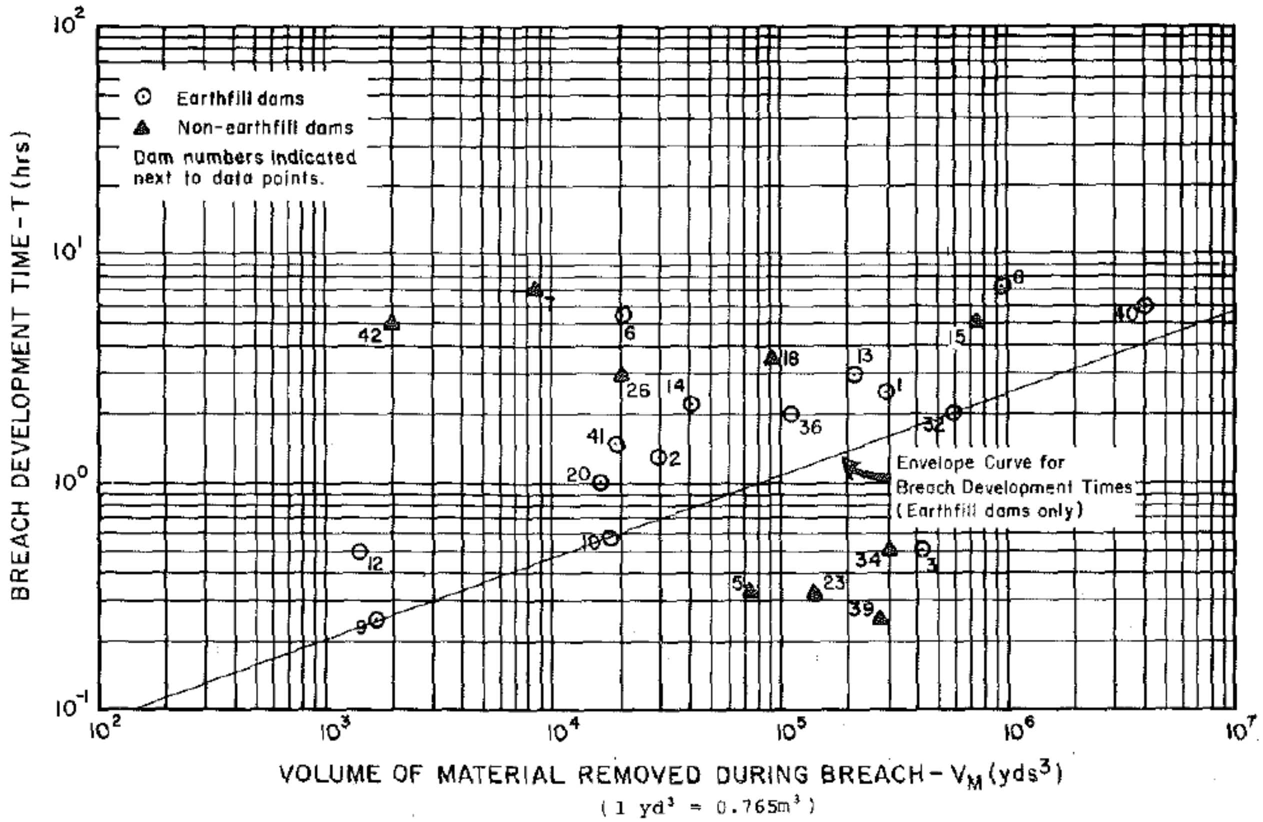


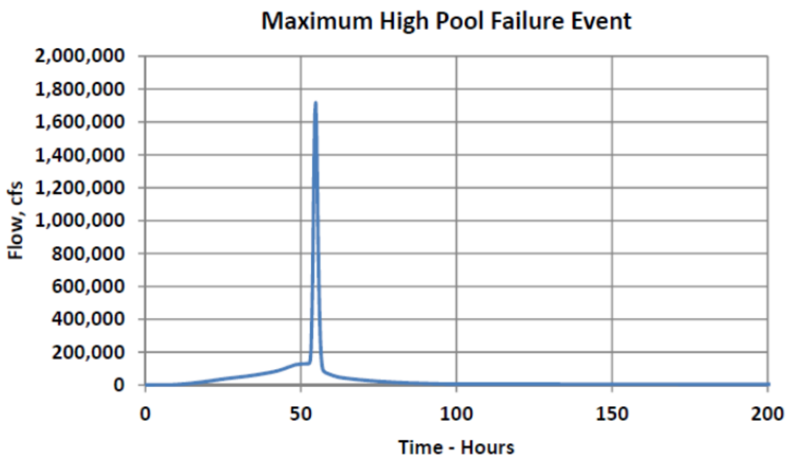
FIG. 2.—Breach Size versus Breach Development Time

MacDonald Langridge-Monopolis 1984

Traditional Methodology – <u>Assume it fails</u>, estimate breach size and then breach formation time.

Based solely on analyses of dams that failed, does not include case histories of dams that overtopped, but didn't fail.

Breach Outflow Hydrograph, Inundated Area, Loss of Life and Property



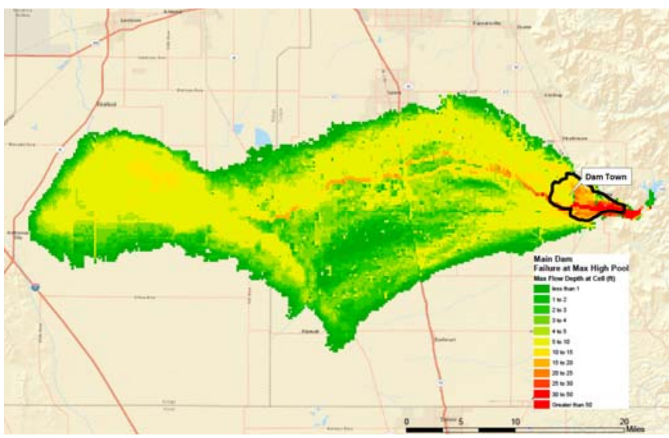


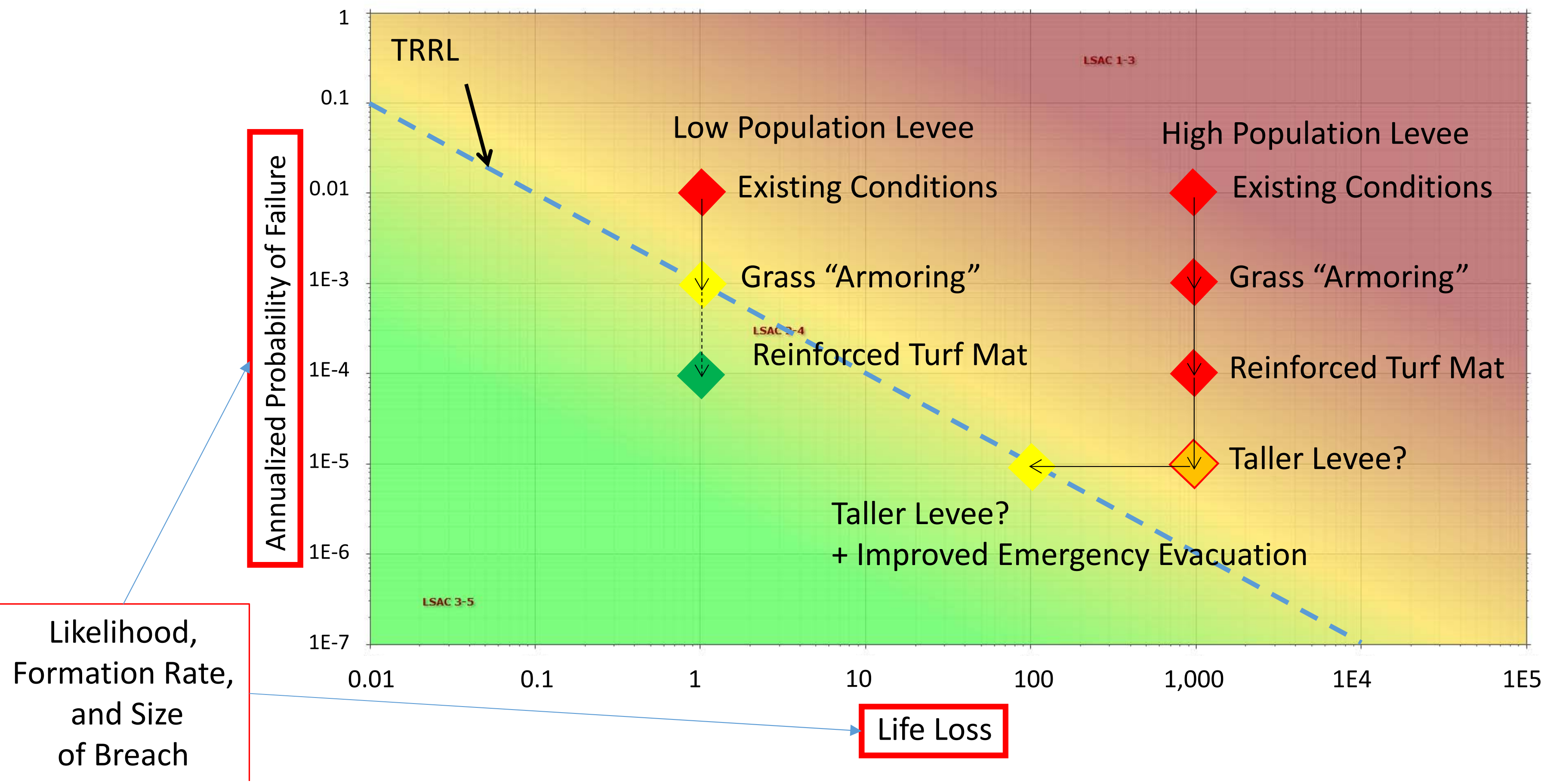
Table 28. Results of the WinDAM B analysis for kd parameters that (1) result in failure, (2) are considered in Distribution 2, and (3) are modeled using a PMF event that results in 6.1 ft of overtopping

kd	Breach Formation Time	Breach Width	Peak Total Outflow (cfs)	Spillway Outflow (cfs)	Incremental Breach Outflow (cfs)
0.094	21.4	478.3	1,795,797	279468.6	1,400,840
0.085	23.5	478.6	1,598,487	279486.1	1,203,530
0.075	26.6	478.7	1,377,938	279489.4	982,981
0.065	30.9	479.5	1,142,044	279489.5	747,087
0.056	36.9	481.2	927,653	279489.5	532,696
0.044	50.9	484.4	643,979	279490	249,022
0.038	65.2	482.7	500,479	279490	105,522
0.035	78	479.8	420,924	279490	25,967
0.032	110.6	473.8	394,960	279490	3
0.008	Non-Breach	7.7	394,957	279489.5	Non-Breach

Table 6-6: Estimated Life Loss by Loading Condition (with Proposed Spillway)

Loading Condition	Daytime			Nighttime		
With Failure	**Min**	**Mean**	**Max**	**Min**	**Mean**	**Max**
1.5 PMF	77	520	1,225	1,243	1,689	2,377
PMF	76	495	1,148	1,061	1,515	2,249
300-yr	34	267	1,223	578	868	2,113
TAS	75	1,388	6,790	578	2,324	8,871
100-yr	504	706	2,835	552	1,392	4,050
No Fail						
1.5 PMF	21	94	201	189	270	392
PMF	10	68	161	155	221	329
300-yr	4	17	33	84	101	119
TAS	0	0	0	0	0	0
100-yr	2	9	20	52	60	74
Incremental						
1.5 PMF	56	426	1,024	1,054	1,419	1,985
PMF	66	427	987	906	1,294	1,920
300-yr	30	250	1,190	494	767	1,994
TAS	75	1,388	6,790	578	2,324	8,871
100-yr	502	697	2,815	500	1,332	3,976
Seismic with Failure						
TAS - Delayed	28	100	206	523	601	721
TAS - Initial	87	536	1,214	587	1,100	1,873
TAS - Immediate	965	1,591	7,475	2,711	5,784	9,814

Risk-Informed Design Progression
Alternatives Analysis



Presentation Topics

- US Army Corps of Engineers Inventory and Performance Challenges
- Flood Risk Management Erosion Issues
    - Risk Analysis – Hazard, Performance and Consequences
    - Potential Failure Modes Analysis and Event Trees
    - Consequences
    - Risk-Informed Design Progression
- Erosion Engineering Needs
    - Models
    - Parameters
        - Critical Shear Stress
        - Erosion Coefficient
        - Wave Overtopping Erosion Thresholds and Rates
- Discussion/Questions

Breach “Initiation” and “Formation” Stages

Figure 12. Generalized description of observed erosion processes during ARS overtopping tests: a) rills and cascade of small overfalls during Stage I, b) consolidation of small overfalls during Stage I, c) headcut at downstream crest, transition from Stage I to Stage II, d) headcut at upstream crest, transition from Stage II to Stage III at breach initiation $t = t_i$, e) flow through breach during Stage III, and f) transition from Stage III to Stage IV at breach formation $t = t_f$.

I. Flow over the embankment initiates at $t = t_0$. Initial overtopping flow results in sheet and rill erosion with one or more master rills developing into a series of cascading overfalls (Figure 12a). Cascading overfalls develop into a large headcut (Figure 12b and 12c). This stage ends with the formation of a large headcut at the downstream crest and the width of erosion approximately equal to the width of flow at the downstream crest at $t = t_1$,

II. The headcut migrates from the downstream to the upstream edge of the embankment crest. The erosion widening occurs due to mass wasting of material from the banks of the gully. This stage ends when the headcut reaches the upstream crest at $t = t_2$ (Figure 12d),

III. The headcut migrates into the reservoir lowering of the crest occurs during this stage and ends when downward erosion has virtually stopped at $t = t_3$ (Figure 12e). Because of the small reservoir size, the peak discharge and primary water surface lowering occurred during this stage, and

IV. During this stage breach widening occurs and the reservoir drains through the breach area (Figure 12f). In larger reservoirs, the peak discharge and primary water surface lowering would occur during this stage ($t_3 < t < t_4$) rather than during stage III. This stage may be broken into two stages for larger reservoirs depending on the upstream head through the breach.

Overtopping Breaching of Noncohesive Homogeneous Embankments (Coleman et al 2002)

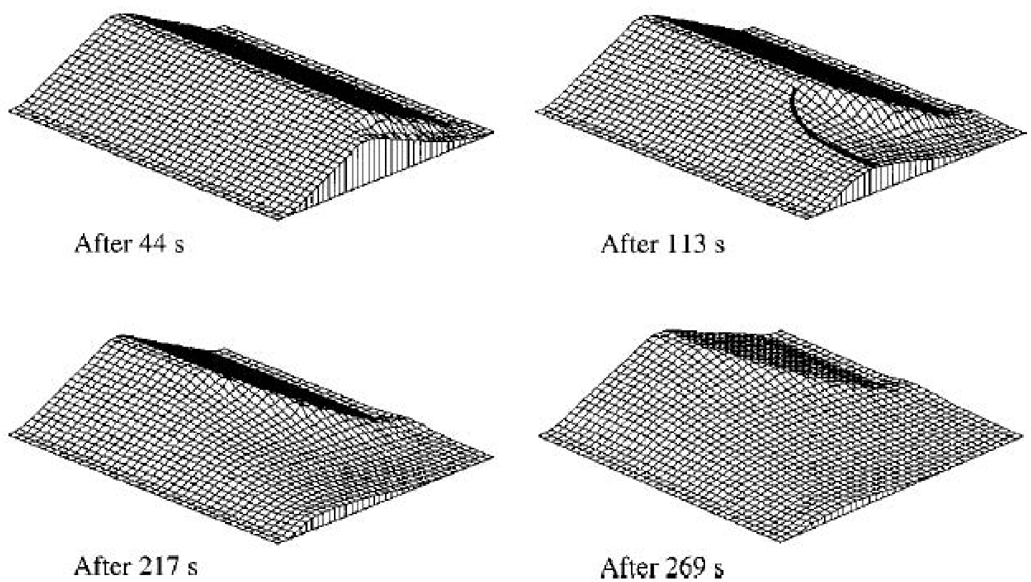


Fig. 2. Breach development for coarse-sand embankment. Curved breach crest line (of length L_b in plan) is highlighted for breach after 113 s

CEATI 2012

RECLAMATION

Managing Water in the West

PAP-1065

Evaluation and Development of Physically-Based Embankment Breach Models

By M.W. Morris, T.L. Wahl, R.D. Tejral, G.J. Hanson, and D.M. Temple

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Hydraulic Investigations and Laboratory Services Group
Denver, Colorado

Evaluation and Development of Physically-Based Embankment Breach Models

M.W. Morris, M.A.A.M. Hassan
HR Wallingford, Wallingford, United Kingdom

T.L. Wahl
Bureau of Reclamation, Denver, Colorado, USA

R.D. Tejral, G.J. Hanson, D.M. Temple
Agricultural Research Service, Stillwater, Oklahoma, USA

ABSTRACT: The CEATI Dam Safety Interest Group (DSIG) working group on embankment erosion and breach modelling has evaluated three physically-based numerical models used to simulate embankment erosion and breach development. The three models identified by the group were considered to be good candidates for further development and future integration into flood modelling software. The evaluation utilized 7 case studies comprising three large-scale tests carried out in Norway (5- to 6-m high embankments); two large-scale tests from the USA (1.75-m high embankments); and the prototype failures of the Oros (Brazil) and Banqiao (China) dams. The breach models evaluated were SIMBA, HR-BREACH, and FIREBIRD BREACH. Results of the evaluation are presented along with details of the continued development of two of the three models (HR BREACH and SIMBA).

1 INTRODUCTION

In 2004 the Dam Safety Interest Group of CEATI International (an international consortium of electric power generating utilities with common research interests) initiated a new research project aiming to advance the state of practice for computer modelling of embankment dam erosion and breach processes. A working group was formed, composed of representatives from CEATI-member utilities with a strong interest in this topic, including several pursuing dam breach modelling research programs of their own. Other organizations with strong research programs on this topic were also invited to join and participate in the working group. The resulting collaboration has brought together many of the most active researchers and organizations working on dam breach modelling worldwide (Table 1).

The working group has pursued this research using a phased approach. The first phase reviewed historical developments related to physical modeling of dam breach processes in laboratory environments (Wahl 2007) and ongoing efforts to develop improved numerical models (Kahawita 2007). Laboratory test data were compiled, especially results from recent, large-scale physical model tests, and real-world case study dam failure data were also collected (Courivaud 2007). The review of numerical models identified three computer models that the working group chose to evaluate in a second phase of the project using the assembled laboratory and real-world case study data sets. Summary results from that evaluation effort are discussed in this paper.

The development and integration of next-generation dam breach modelling tools into dynamic flood routing models and the continued improvement of those models going forward is the long-term objective of the CEATI-sponsored project. The models studied thus far are focused primarily on the overtopping[1] failure mode and relatively simple embankment geometries, but development is underway on modules to simulate internal erosion and more complex embankment geometries. These capabilities are expected to continue to improve over time.

Table 1. — Members of the CEATI Working Group, and other project sponsors.

ORGANIZATION	ROLES	Primary Representatives
CEATI International	Technical coordination	Gary Salmon (deceased)
Electricité de France	Assemble case studies of real dam failures. Erodimeter and piping erosion research.	Jean-Robert Courivaud
Hydro Québec / École Polytechnique Montréal	Review of numerical models for simulating dam breach, development of FIREBIRD BREACH model.	Tai Mai Phat, Réne Kahawita
Bureau of Reclamation	Review of laboratory physical hydraulic modelling programs. Investigation of erodimeters.	Tony Wahl
USDA-Agricultural Research Service	Large-scale laboratory testing and development of SIMBA/WinDAM models. Development and investigation of erodimeters.	Greg Hanson, Ron Tejral, Darrel Temple
HR Wallingford	Small- and large-scale physical model testing (IMPACT project), developers of HR-BREACH model	Mark Morris, Mohamed Hassan
US Army Corps of Engineers	Erodimeter evaluation, breach model evaluation, potential integration of breach modelling technology into HEC-RAS suite	Jeff McClenathan, Johannes Wibowo, Michael Gee
Elforsk, Energo Retea	Numerical breach model evaluation	Aecila Romanas, Fredrik Persson
Ontario Power Generation	Numerical breach model evaluation	Allan Kirkham, Yibing Zhang
Other sponsors: Churchill Falls Hydro, EoN Vasserkraft, Great Lakes Power, Manitoba Hydro, New York Power Authority, Seattle City Light, Scottish & Southern Energy		

[1] In this paper, the term 'overtopping' is used to mean the continuous overflow of water rather than wave overtopping.

Model Comparisons

	HR-BREACH	SIMBA / WinDAM	FIREBIRD	NWS-BREACH
Erosion Process Models	Good	Good	Fair	Limited
Surface protection	Vegetation (CIRIA) and rip-rap	Vegetation, riprap in WinDAM	Limited	Yes
Headcut erosion	Good	Best	No	No
Stress-based	—	Yes	—	—
Energy-based	Yes	Yes (in WinDAM)	—	—
Surface erosion	Yes	No	Yes	Yes
Mass-wasting / soil-wasting	Stress-based bank failures and arch failure	Bank failures implicit	Some	Some
Effects of Submergence	Yes	Yes (in WinDAM)	No	Yes
Piping progression	Yes	In development	Some	Yes
Data Input Guidance	Good	Good	Limited	Limited
Ease of Use	Good	Good	Difficult	Difficult
Computational Efficiency	Good	Good	Fair	Good
Documentation	Excellent	Excellent	Limited	Good
Organizational Support for Continued Development	Good	Good	Weak	None
Embankment Geometry Options	Simple Zoning	Homogeneous, (Zoned in future)	Simple Zoning	Primitive Zoning

Presentation Topics

- US Army Corps of Engineers Inventory and Performance Challenges
- Flood Risk Management Erosion Issues
  - Risk Analysis – Hazard, Performance and Consequences
  - Potential Failure Modes Analysis and Event Trees
  - Consequences
  - Risk-Informed Design Progression
- Erosion Engineering Needs
  - Models
  - Parameters
    - Critical Shear Stress
    - Erosion Coefficient
    - Wave Overtopping Erosion Thresholds and Rates
- Discussion/Questions

Soil Erosion Model

10.3 Evaluation of Surficial Current and Wind/Wave Action Erosion.

10.3.1 Several erosion studies have been performed that focus on identifying the erosion parameters and correlating those parameters to formulate an expression (a physical model) for erosion rates (Hanson and Temple, 2001; Hanson and Cook, 2004). The governing equation for this model is:

$$\dot{\varepsilon} = \left(k(\tau - \tau_c)\right) \quad (10\text{-}1)$$

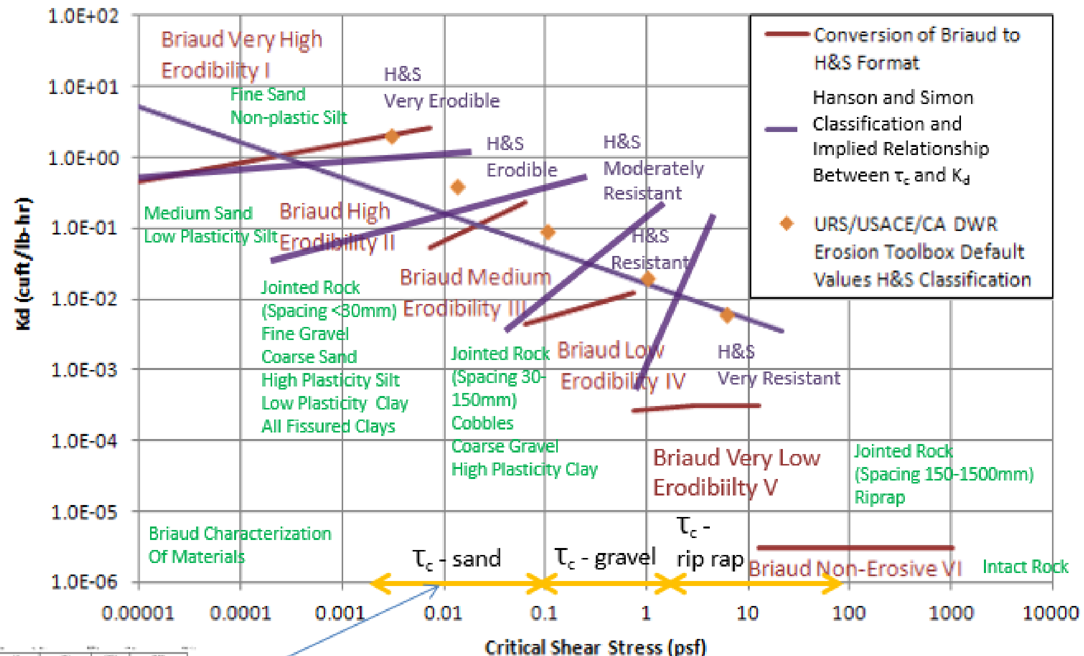
(10-1)

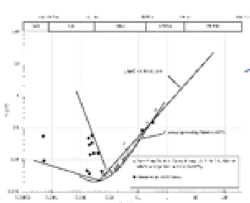
where

$\dot{\varepsilon}$ = erosion rate
k= erodibility coefficient or detachment rate coefficient (ft^3/lb-hr)
τ= effective hydraulic stress on the soil boundary (lb/ft^2)
τ_c= critical shear stress (lb/ft^2), i.e. the shear stress at which erosion starts

10.3.2 The erosion rate ($\dot{\varepsilon}$) is a function of both hydraulic (τ) and geotechnical (k, τ_c) parameters. Effective hydraulic stress (τ) mainly depends on characteristics of water-soil boundary, current/stream velocity and/or wind wave height and period. Both k and τ_c are functions of the engineering properties of the levee and the foundation materials. The following sections describe the hydraulic and geotechnical parameters in the above model.

“Hanson” erosion resistance, “Briaud” erodibility, and Levee Erosion Toolbox (URS 2007) default values for k_d and associated τc for the various “Hanson” erosion resistance classifications and Shield’s Diagram τc from Briaud (2001) to be cited as the primary source for analysis parameters in Engineering Manual 1110-2-1913.





τ_c versus D_{50} for “clean” materials based on Shield’s Diagram from Briaud 2001

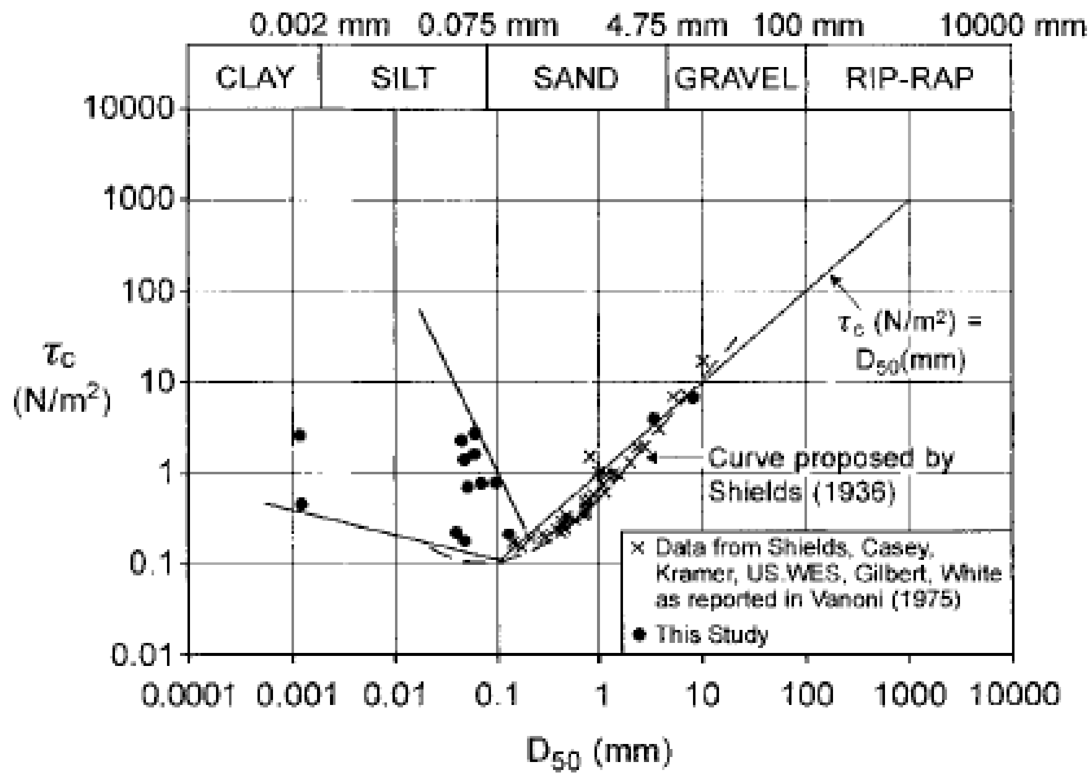
Presentation Topics

- US Army Corps of Engineers Inventory and Performance Challenges
- Flood Risk Management Erosion Issues
  - Risk Analysis – Hazard, Performance and Consequences
  - Potential Failure Modes Analysis and Event Trees
  - Consequences
  - Risk-Informed Design Progression
- Erosion Engineering Needs
  - Models
  - Parameters
    - Critical Shear Stress
    - Erosion Coefficient
    - Wave Overtopping Erosion Thresholds and Rates
- Discussion/Questions

Factors Likely Affecting Critical Shear Stress

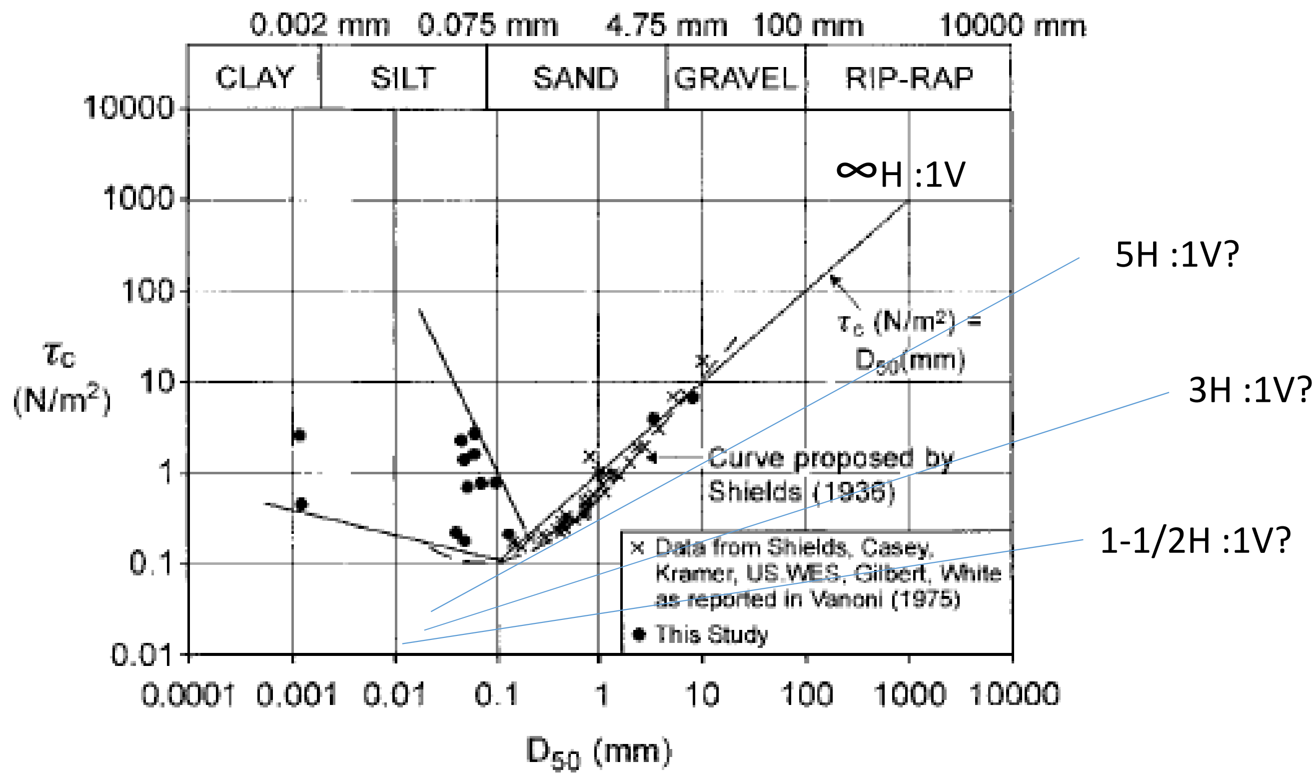
- Slope of Eroding Surface
- Cementation
- Compaction
  - Compaction Energy Level
  - Moisture Content
- Consolidation
- Age?

Uncemented, Normally Consolidated Materials
Critical Shear Stress – Horizontal Flow



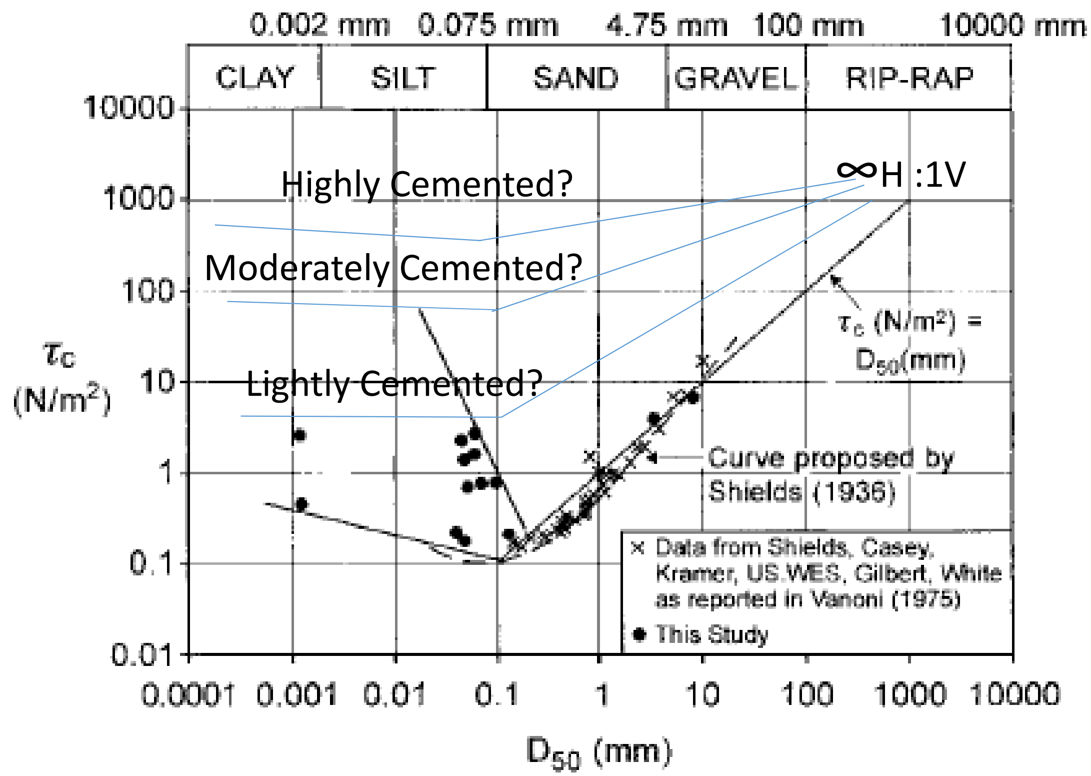
Uncemented Materials

Critical Shear Stress – Decreases with Increased Slope of Eroding Surface

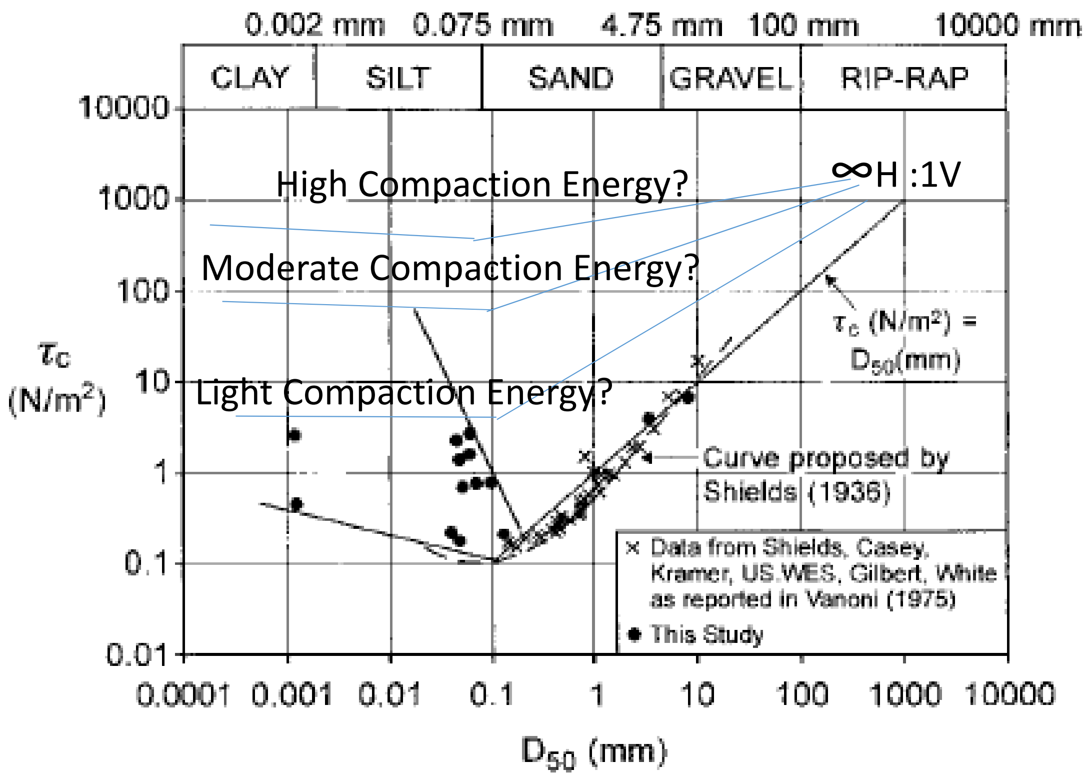


Cemented Materials

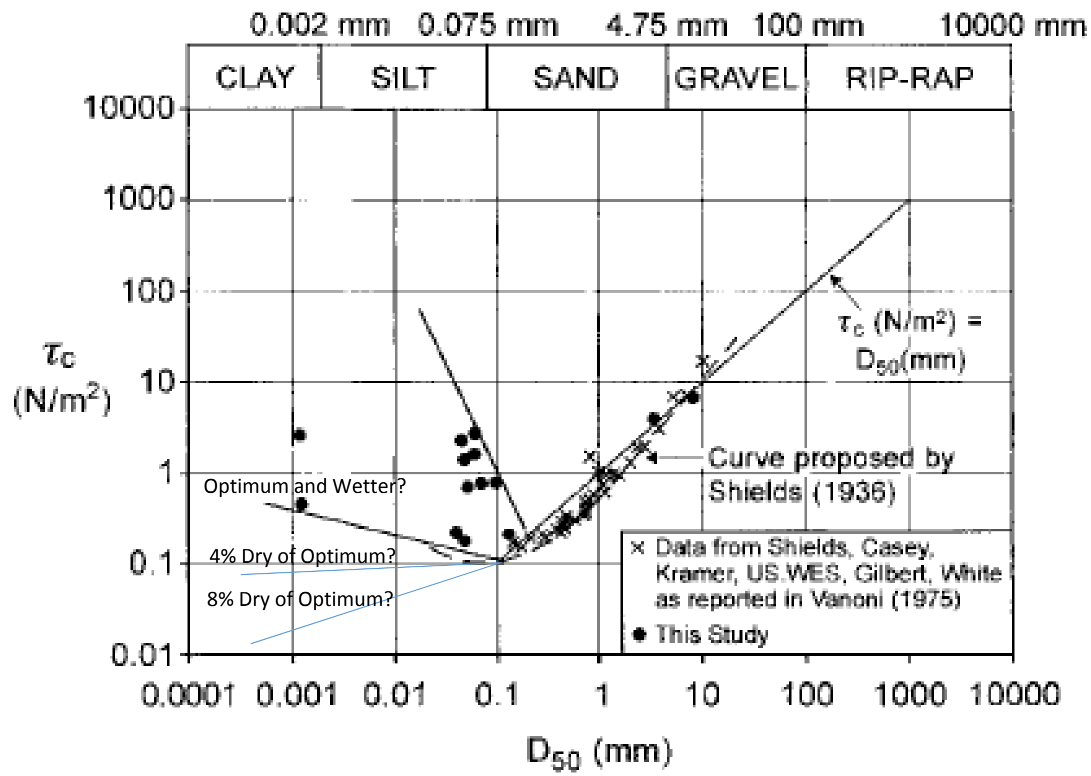
Critical Shear Stress – Increases with Cementation



Compacted Materials
Critical Shear Stress – Increases with Compaction Effort

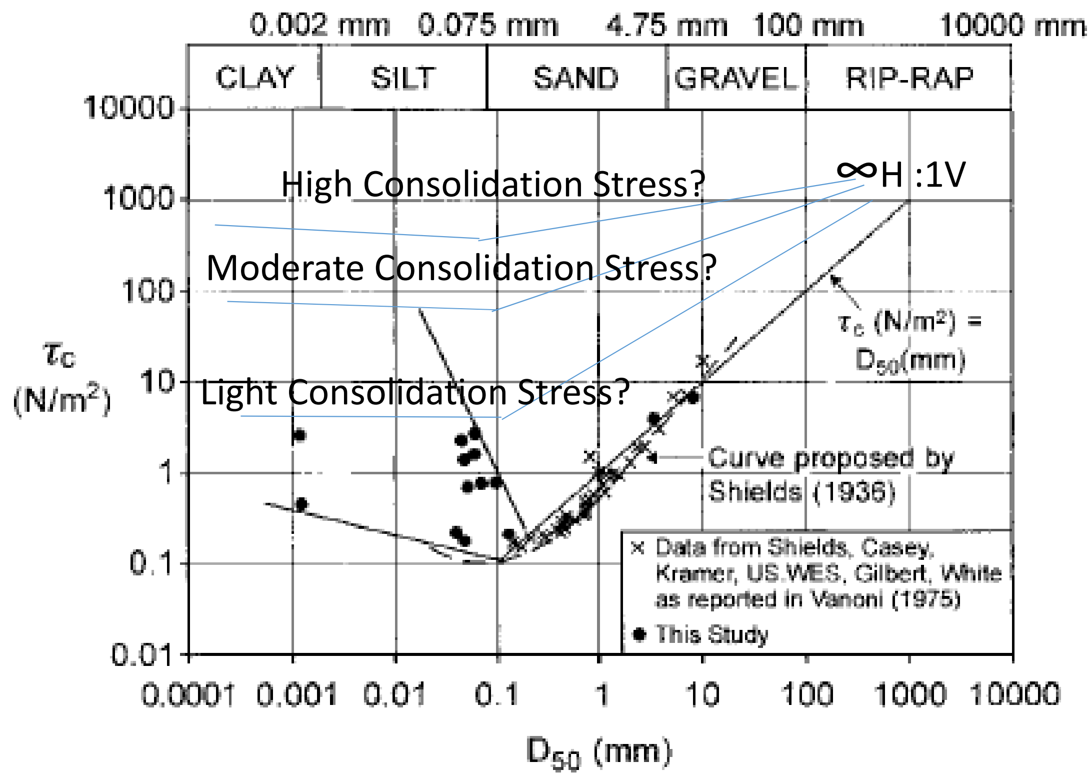


Compacted Materials
Critical Shear Stress – Increases up to Optimum Compaction Water Content

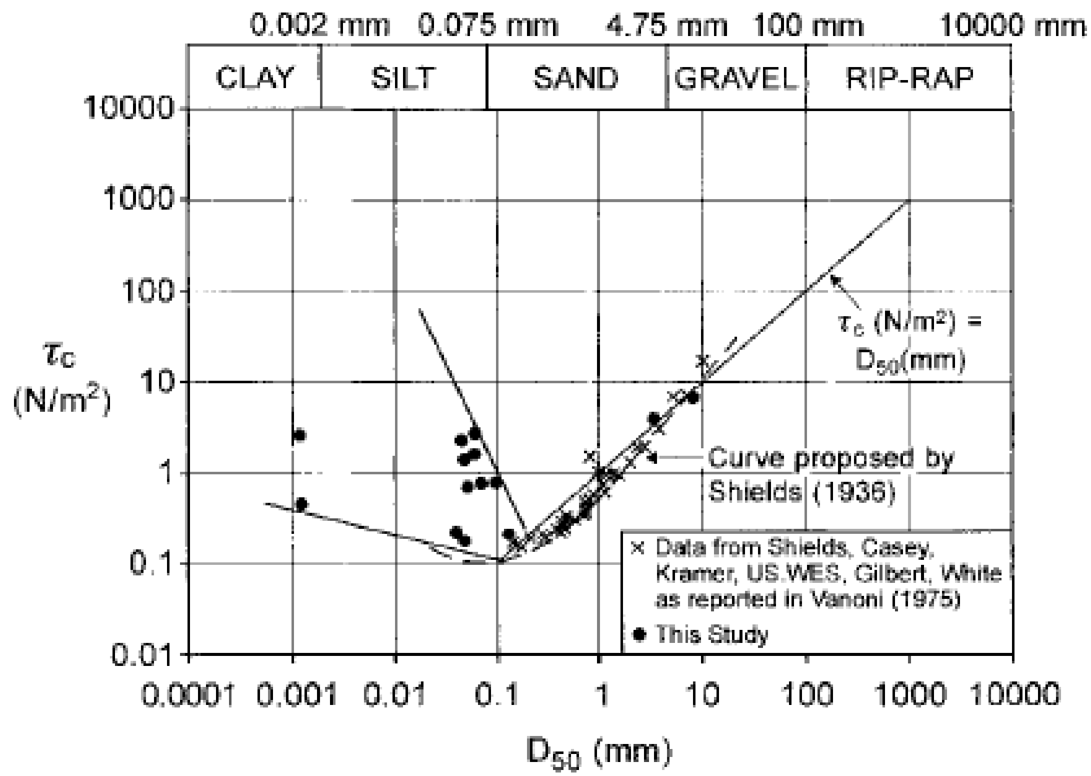


Consolidated Materials

Critical Shear Stress – Increases with Maximum Past Pressure



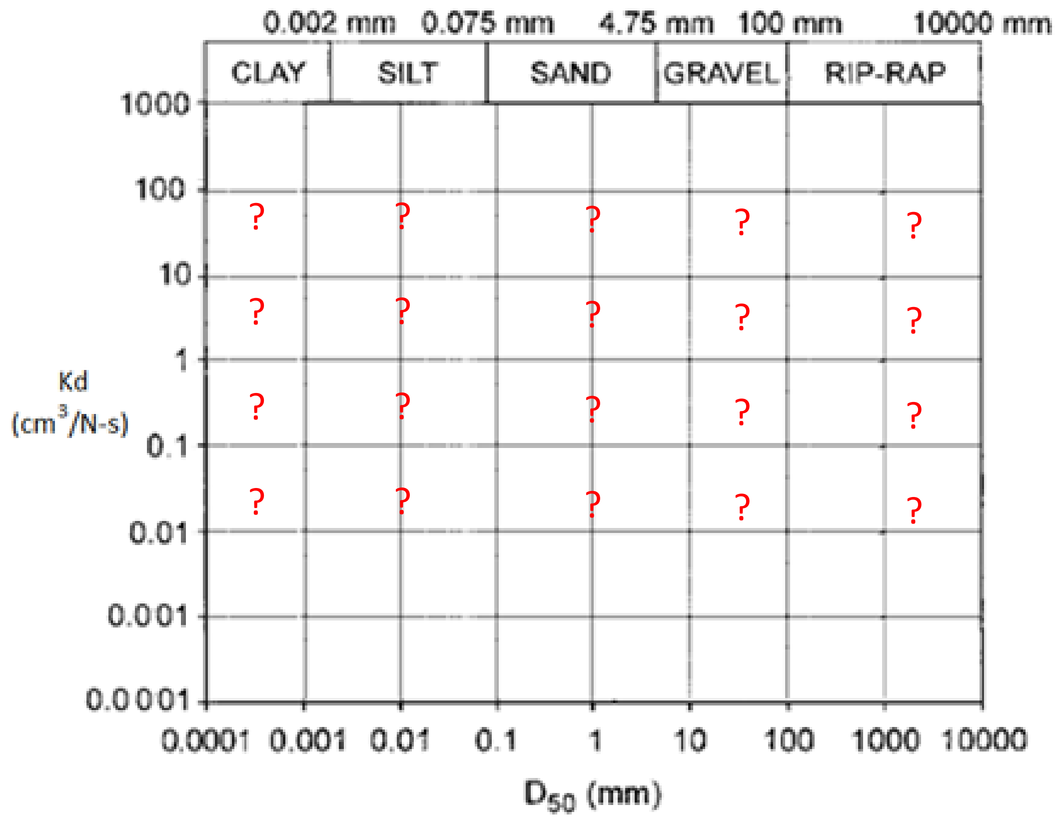
Changes with Time?
Critical Shear Stress – Horizontal Flow



Presentation Topics

- US Army Corps of Engineers Inventory and Performance Challenges
- Flood Risk Management Erosion Issues
  - Risk Analysis – Hazard, Performance and Consequences
  - Potential Failure Modes Analysis and Event Trees
  - Consequences
  - Risk-Informed Design Progression
- Erosion Engineering Needs
  - Models
  - Parameters
    - Critical Shear Stress
    - Erosion Coefficient
    - Wave Overtopping Erosion Thresholds and Rates
- Discussion/Questions

Are There Similar Variations in K_d with Variations In the Same Factors?



Variations in Soil Type and Compaction Moisture Content

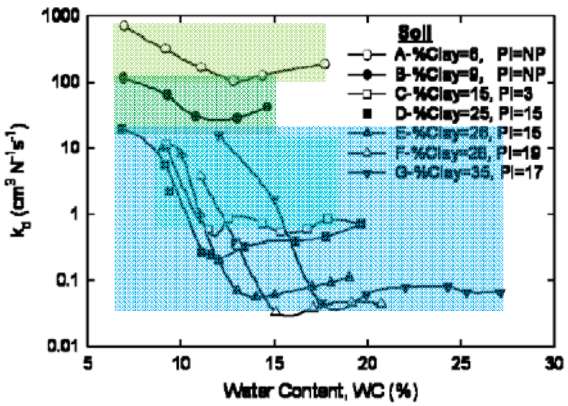
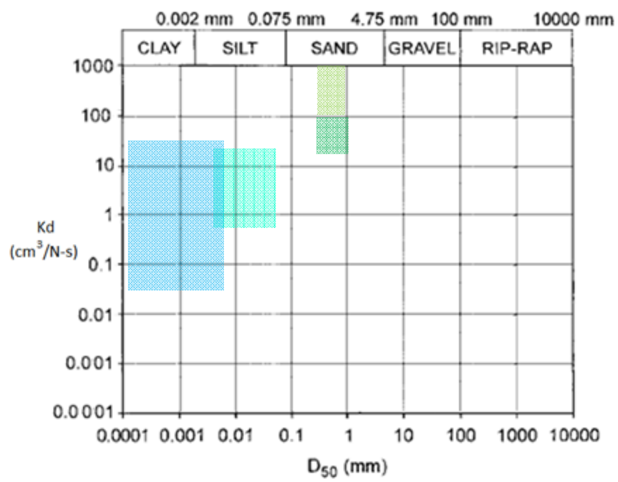


Figure 4. Resulting k_d curves for soils tested at USDA-ARS HERU at standard compaction effort, 6.0 kg-cm/cm^3.

Table 1. Property of tested soils at the USDA-ARS Laboratory.

Soil Sample Designation	USCS Classification	Atterberg Limits: Liquid Limit(%)	Atterberg Limits: Plasticity Index (%)	Texture: % Sand >0.074 mm	Texture: % Clay < 0.002 mm
A	SM	NP	NP	73	6
B	SM	NP	NP	64	9
C	ML	23	3	32	15
D	CL	26	15	35	25
E	CL	31	15	24	26
F	CL	37	19	20	28
G	CL	37	17	13	35



Hanson et al 2010

Variations in Compaction Effort and Moisture Content

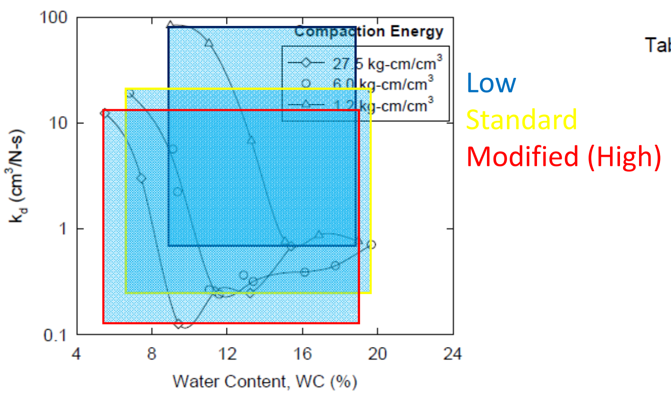
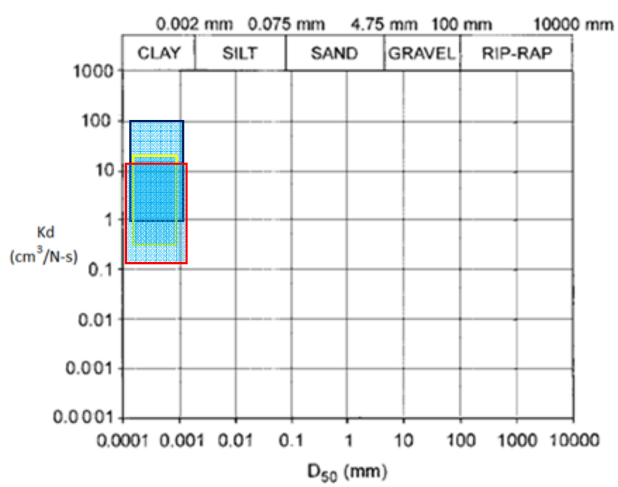


Table 1. Property of tested soils at the USDA-ARS Laboratory.

Soil Sample Designation	USCS Classification	Atterberg Limits: Liquid Limit(%)	Atterberg Limits: Plasticity Index (%)	Texture: % Sand >0.074 mm	Texture: % Clay < 0.002 mm
A	SM	NP	NP	73	6
B	SM	NP	NP	64	9
C	ML	23	3	32	15
D	CL	26	15	35	25
E	CL	31	15	24	26
F	CL	37	19	20	28
G	CL	37	17	13	35



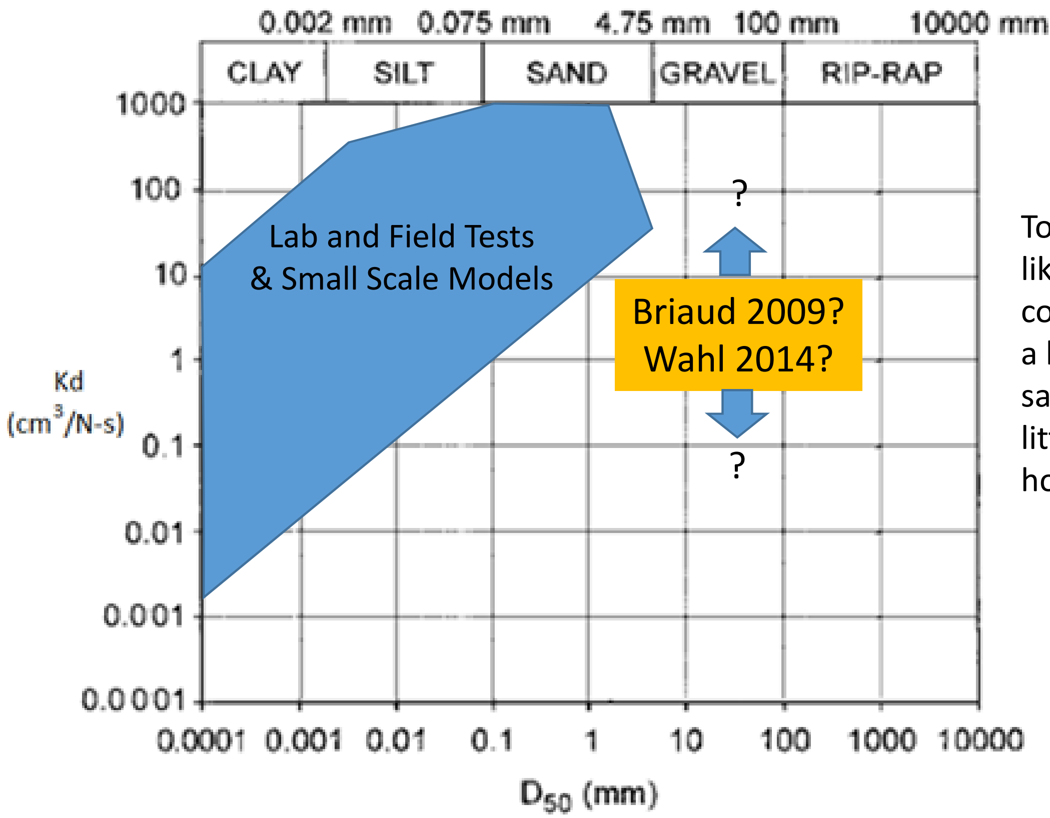
Hanson et al 2010

We have data supporting Kd values for Clay and Silt, but not for Gravel.

Does an inclined gravel have a K_d like an inclined sand?

Does a <u>clean</u> gravel have a K_d like sand or like clay?

Does a <u>clayey</u> gravel have a K_d like sand or like clay?

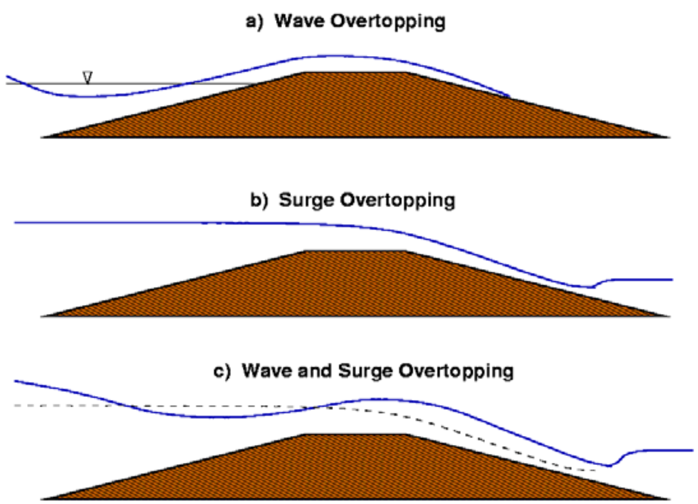


To many, it seems likely that gravel and cobbles would have a lower Kd than sands, but we have little data to support how much lower.

Presentation Topics

- US Army Corps of Engineers Inventory and Performance Challenges
- Flood Risk Management Erosion Issues
  - Risk Analysis – Hazard, Performance and Consequences
  - Potential Failure Modes Analysis and Event Trees
  - Consequences
  - Risk-Informed Design Progression
- Erosion Engineering Needs
  - Models
  - Parameters
    - Critical Shear Stress
    - Erosion Coefficient
    - Wave Overtopping Erosion Thresholds and Rates
- Discussion/Questions

Transient Wave Loading



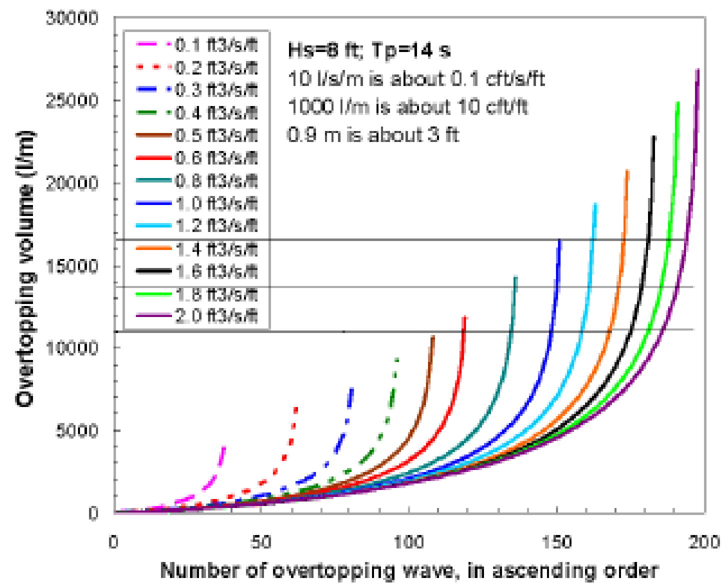
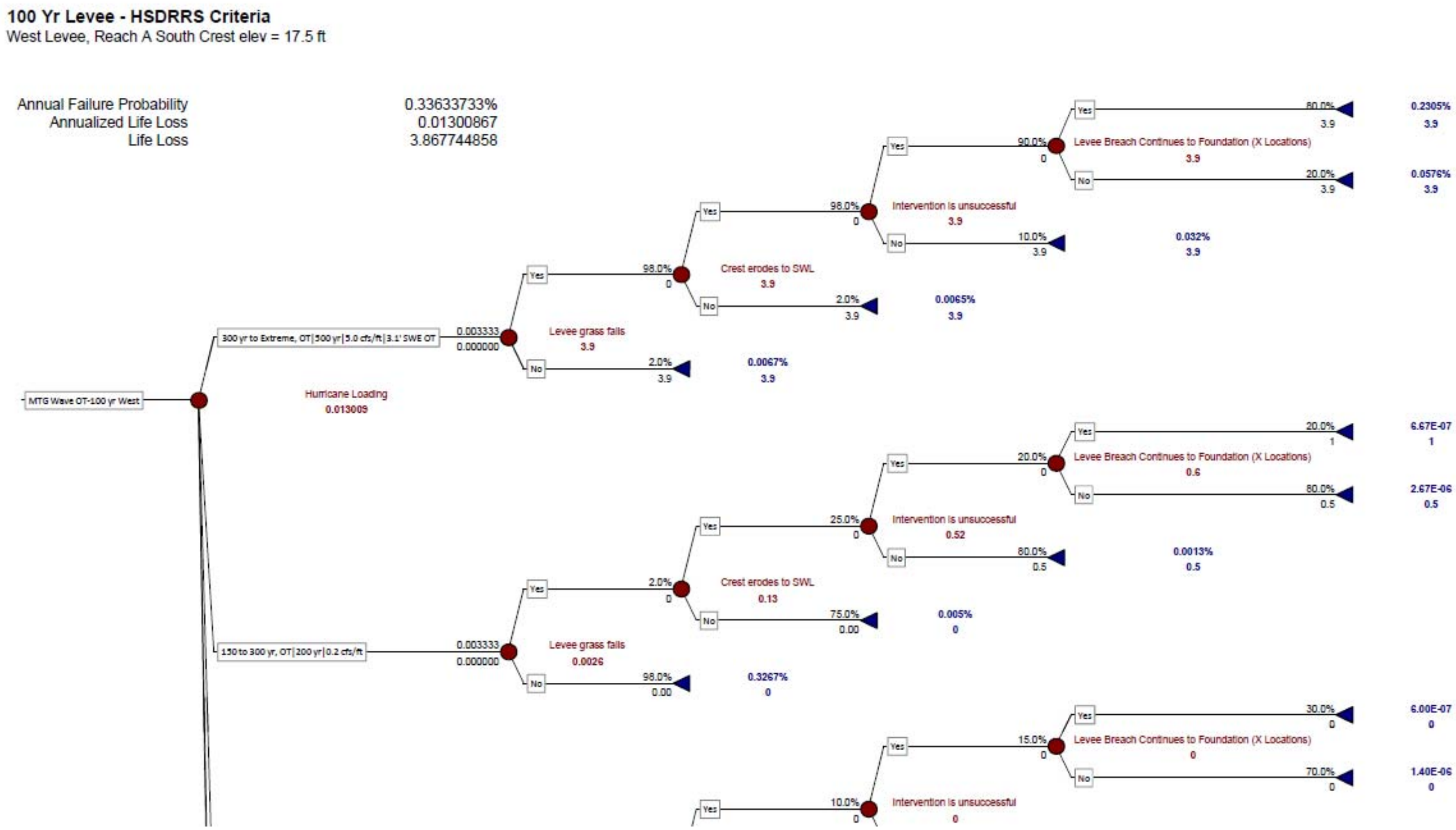
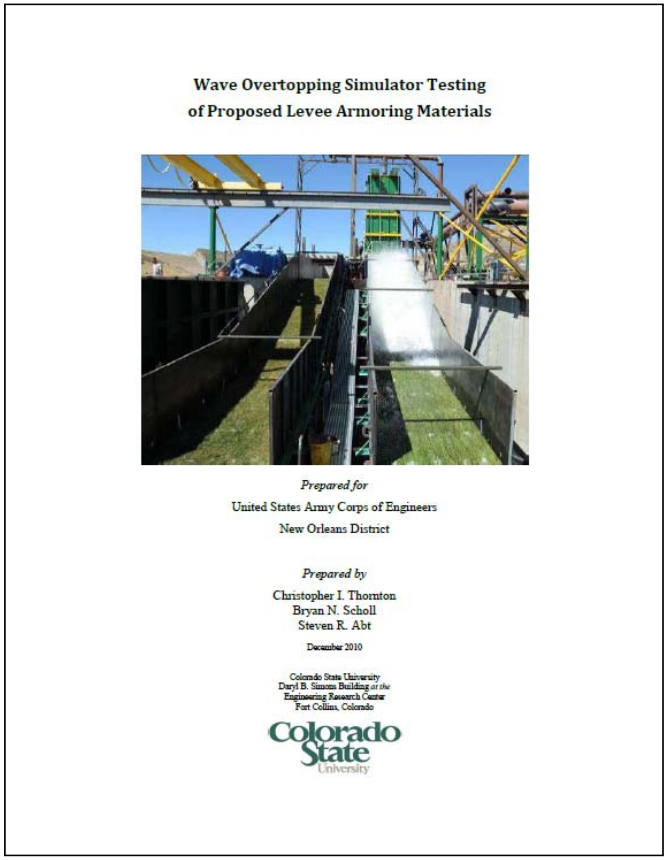


Figure 2.3. Required distribution of overtopping volumes for H_{m0} = 8 ft with T_p = 14 s.

Overtopping Breach Event Trees



Estimation of Erosion Rates Based On CSU Flume Test Results


Wave Overtopping Simulator Testing
of Proposed Levee Armoring Materials

Prepared for
United States Army Corps of Engineers
New Orleans District

Prepared by
Christopher I. Thornton
Bryan N. Scholl
Steven R. Abt

December 2010

Colorado State University
Daryl B. Simons Building *at the*
Engineering Research Center
Fort Collins, Colorado

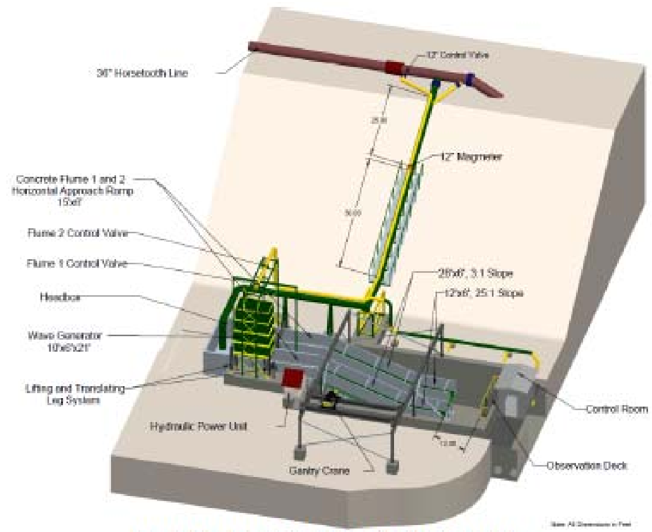


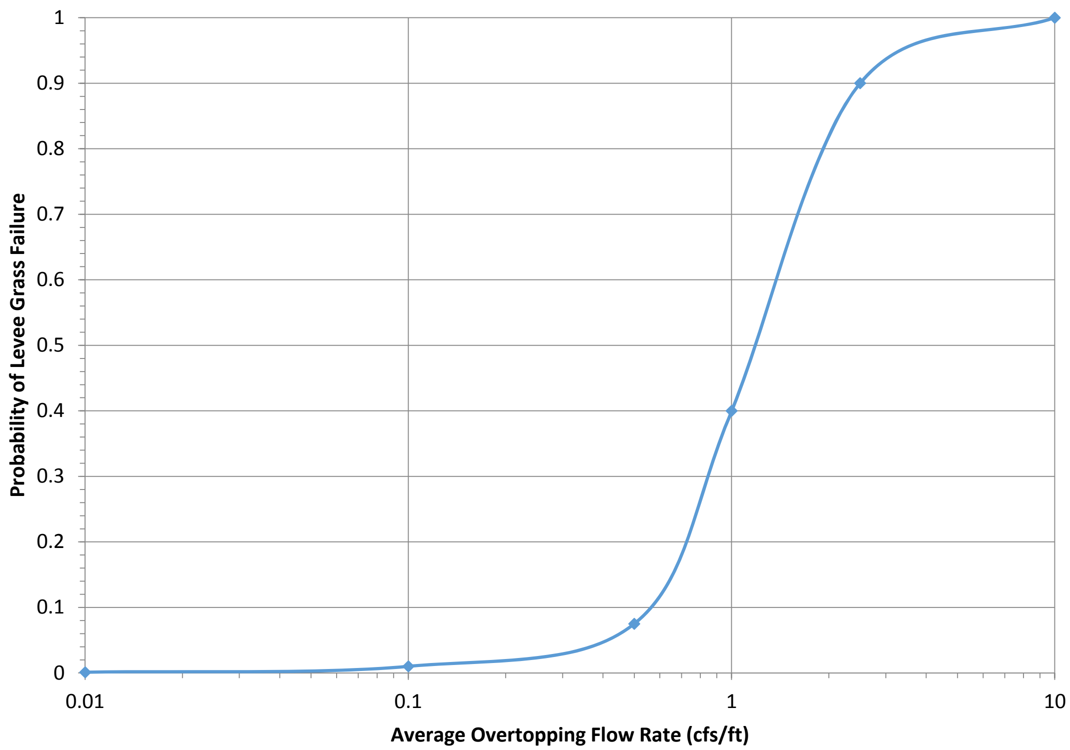
Figure 2-4. Wave overtopping facility schematic

Levee Grass Armoring Fails

Event Information	
Overtopping Flow Rate:	0.01 cfs/ft
Type of Loading:	Constant overtopping flow
Influence Factors	
More Likely Factors	**Less Likely Factors**
• Can't count on 100% grass coverage due to salinity in the levee environment • No case histories of actual levee performance • If levee materials contain silt or sand, their erosion resistance would be reduced • Levees could contain man-made or animal defects that could lead to poor performance	• 0.01 cfs/ft is less than European standard design flow for sandy sites (typically as high as ~0.1 cfs/ft) • The levee embankment soils will be clay which is expected to have a tolerable flow rate on the order of 0.1 cfs/ft • Vietnamese case histories indicate Bermuda Grass slopes can begin to sustain damage at overtopping rates of 0.5 to 0.7 cfs/ft (Trung et al, 2010 and Trung et al, 2011) • USCS allows grass-lined channel velocities of up to 5 ft/sec (USDA-SCS, 1984) • New Orleans District earthen channels are designed for velocities of less than 3 ft/sec • The Netherlands model studies showed that nominal grass cover can withstand up to 0.54 cfs/ft (Wise, 2010) • CSU model studies showed that Bermuda Grass with exceptionally high root density did not fail at flow rates of approximately 4 cfs (Thornton et al, 2010)

Event Information	
Overtopping Flow Rate	1.0 cfs/ft
Type of Loading:	Constant overtopping flow
Influence Factors	
More Likely Factors	**Less Likely Factors**
• Can't count on 100% grass coverage due to salinity in the levee environment • No case histories of actual levee performance • If levee materials contain silt or sand, their erosion resistance would be reduced • Levees could contain man-made or animal defects that will lead to poor performance • 0.1 cfs/ft is in the maximum range of the European standard design flow for sandy sites • The soils are clay here which is expected to have a tolerable flow rate on the order of 0.1 cfs/ft • Vietnamese case histories indicate Bermuda Grass slopes can begin to sustain damage at overtopping rates of 0.5 to 0.7 cfs/ft (Trung et al, 2010 and Trung et al, 2011) • The Netherlands model studies showed that nominal grass cover can withstand 0.6 cfs/ft (reference, year)	• USCS allows grass-lined channel velocities of up to 5 ft/sec (USDA-SCS, 1984) • New Orleans District earthen channels are designed for velocities of less than 3 ft/sec • CSU model studies showed that Bermuda Grass with exceptionally high root density did not fail at flow rates of approximately 4 cfs (Thornton et al, 2010) • The levees are anticipated to be entirely composed of clay without a sand core

Probability Grass Armor Fails as a Function of Average Overtopping Flow Rate



Estimate of Bare Soil Erosion Rates

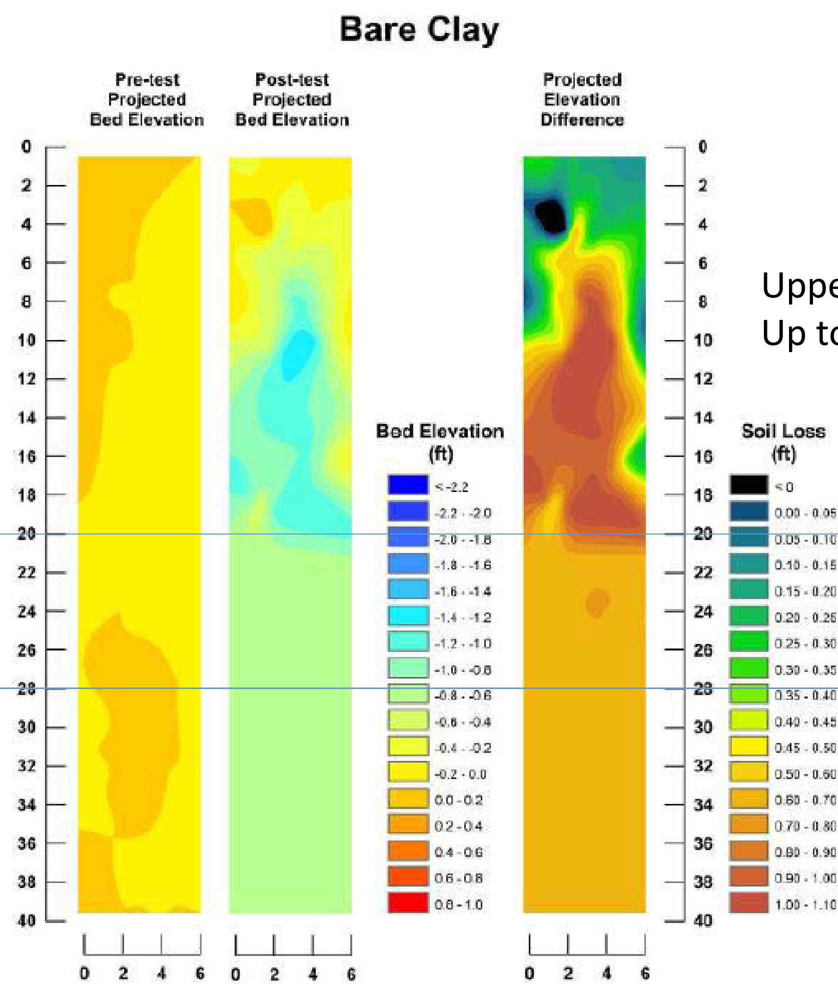
Figure 3-2. Bare clay soil test installation

Up to 1' erosion after 1:00?

a. Bare clay soil slope, upper 20 ft

b. Bare clay soil slope, lower 20 ft

Figure 3-3. Soil surface following first hour of bare soil testing



Upper Section
Up to 1’ erosion after 1:20

Smoothing
Transition?

Bend

Lower Section
~0.6’ to 0.7 after 1:20

Approximate erosion loss rates

Bare Clay	q_{wave} = 0.1 cfs/ft	q_{wave} =0.2 cfs/ft
Upper Steep Slope	<1 ft / hour (apparent problem with concentration at transition)	Slightly more (transition problem
Lower Flatter Slope	<0.1 ft / hour	~1.95 ft / hour

Presentation Topics

- US Army Corps of Engineers Inventory and Performance Challenges
- Flood Risk Management Erosion Issues
  - Risk Analysis – Hazard, Performance and Consequences
  - Potential Failure Modes Analysis and Event Trees
  - Consequences
  - Risk-Informed Design Progression
- Erosion Engineering Needs
  - Models
  - Parameters
    - Critical Shear Stress
    - Erosion Coefficient
    - Wave Overtopping Erosion Thresholds and Rates
- Discussion/Questions