Upcoming Changes for EM 1110-2-1913
Design, Construction, and Evaluation of Levees

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USSD Fall 2015 Levee Workshop

November 3, 2015
Objective

- To provide an overview of the updates being proposed for EM 1110-2-1913 Design, Construction, and Evaluation of Levees
Contents

- Introduction and overview of some regional design differences
- Review of Chapters
- Questions and Discussion
Complaint About Old EM

“It’s great if you are from the Mississippi Rivers and Tributaries Project, but there’s nothing about the types of levee designs we do here in ______________.”
Primary Objectives

To update the USACE EM for Design, Construction and Evaluation of Levees

► Compile information on regional design approaches and performance expectations including operations, maintenance, flood-fighting efforts and associated documentation

► Use risk-based potential failure modes to establish design objectives and approaches

► Assess adequacy of traditional design with risk analysis methods
# EM Revision Team

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<tr>
<td>1 Scott Shewbridge</td>
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<tr>
<td>2 Noah Vroman</td>
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<tr>
<td>5 Neil Schwanz</td>
</tr>
<tr>
<td>6 Tom Brandon – Adjunct to ERDC</td>
</tr>
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| MVD - Mississippi Valley Division MVK - Vicksburg |
| ERDC                                             |
| MVD - Mississippi Valley Division MVS - St. Louis |
| MVD - Mississippi Valley Division MVP - St. Paul |
| Virginia Tech                                  |

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## Regional Representatives “EM Collaborators”
### Regional Levee Design PFMA Session / ATR

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<tr>
<td>1 Pat Conroy Geotech MVD - Mississippi Valley Division - St. Louis</td>
</tr>
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<td>2 Doug Chitwood, PE GE Geotech SPD - South Pacific Division - Los Angeles</td>
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<td>6 Tom Mack Chief Geotech MVD - Mississippi Valley Division - Rock Island</td>
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<tr>
<td>7 Mark Woodward Geotech MVD - Mississippi Valley Division - New Orleans</td>
</tr>
<tr>
<td>8 James Snyder Geotech NAD - North Atlantic Division - Baltimore</td>
</tr>
</tbody>
</table>

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6/81
Example Case Histories
Mississippi Valley Division – New Orleans

WHERE WE ARE – U.S. ARMY CORPS OF ENGINEERS

Other Organizations
- Engineer Research and Development Center (ERDC)
- Engineering and Support Center, Nashville
- Army Geospatial Center
- U.S. Army Engineer Waterways Experiment Station
- Marine Corps
- Institute for Water Resources
- TEA, Engineer Research
Marchand Levee Failure 1983
Flow Slides in Sand

River Stage

Resuspension/Transport

T=0

RESEDIMENT DURING T=t_1 TO T=t_2

RESERDIMENT DURING T=0 TO T=t_1

c. TIME T=t_2 RETROGRESSION CONTINUES

T=t_2

OVERBURDEN

ZONE A SANDS

Resuspension/Transport

T=t_1

RESEDIMENT DURING T=t_2 TO T=t_3

d. TIME T=t_3 OVERBURDEN UNDERCUT BEGINS, BLOCKS FAIL AND RIDE OUT UPON FLUIDIZED/LIQUIFIED LAYER

ζ LEVEE

ζ LEVEE
ACM provides the best level of protection along the rivers in the New Orleans District’s jurisdiction due to the depth of water and construction issues with using other materials in those depths.
Hurricane Protection – Stability
With Reinforcement Fabric

Add 15 ft minimum for "Veg Free Zone" plus construction and maintenance zone

FLOOD SIDE = NEW PS

C/L

113.6 FT

138.6 FT

Add 15 ft minimum for "Veg Free Zone" plus construction and maintenance zone

PROTECTED SIDE = NEW FS

1.58

Reinforcement Fabric @ EL 2
125 foot length
50 feet on NEW PS
75 feet on NEW FS
Fabric with 1400 lbs/in @ 5% Strain

LGM - Levee Reach C-Nort
Geotextile Installation
Example Case Histories
Mississippi Valley Division – Rock Island

WHERE WE ARE – U.S. ARMY CORPS OF ENGINEERS
Design of Sand Levees

Design Philosophy

• The through seepage control must prevent the slope failure and excessive erosion of the landside slope.

• The under seepage design criteria must control the hydraulic gradients landward of the levee to prevent piping and excessive uplift pressures on the landside impervious stratum.

• Stability usually not an issue
Drury and Iowa River/Flint Creek Test Sections 1962 and 1964
Results of Test Section: Typical Sand Levee Cross Section

- 4:1 Riverside slope
- 10 foot crown
- 5:1 Land side slope
  - Steeper slopes resulted in erosion due to through-seepage.
- 10 h base width (The width at the base of the levee is 10 time the height of the levee.)
  - On heights greater than 10 feet a berm is required
    - 3 feet thick usually 20 feet wide.
    - Design based on seepage and stability analysis (EM 1913)
Example Case Histories
Northwestern Division – Kansas City

WHERE WE ARE – U.S. ARMY CORPS OF ENGINEERS

Other Organizations
- Engineer Research and Development Center (ERDC)
- Corps of Engineers
- North American River Basins
- South Pacific Division
- Great Lakes & Ohio River Division
- South Atlantic Division
- Transatlantic Division
- Middle East District
- Afghanistan Engineer Command

LEGEND
- Army Research
- Environment Protection
- Water Resources
- Corps Management
- Army Executive

Northwestern Division
South Pacific Division
Great Lakes & Ohio River Division
South Atlantic Division
Transatlantic Division
Middle East District
Afghanistan Engineer Command
Example Case Histories
North Atlantic Division – Baltimore
Typical Levee Section (with MSE Wall)
Example Case Histories
South Pacific Division – Los Angeles

WHERE WE ARE – U.S. ARMY CORPS OF ENGINEERS
Site Geology
Levee Embankment – Soil Cement

Typical Soil Cement Embankment Section

Original Levee

Channel Grade

Soil Cement

Original Levee

Channel Grade

Soil Cement

Original Levee

Landside Slope
Example Case Histories
South Pacific Division – Sacramento
Design Concept: Deep Cutoff

- Degrade ½ levee height
- Extend cutoff wall down into low perm layer
- Regrade with clay core
Example Case Histories

Good Coverage for Entire Country

WHERE WE ARE – U.S. ARMY CORPS OF ENGINEERS

Other Organizations
- Engineer Research and Development Center (ERDC)
- Engineering and Support Center, Huntsville
- Army Geospatial Center
- U.S. Army Engineer District
- Marine Corps Center
- Institute for Water Resources
- JFCS, Engineer Waterways
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- Chapter 2. Levee Failure Modes and Risk-Based Decisions During Design and Construction
- Chapter 3. Field Investigations for Levees
- Chapter 4. Laboratory Testing for Levees
- Chapter 5. Borrow Areas
- Chapter 6. Subsurface Interpretation
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- Chapter 9. Settlement
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- Appendix D. Subsurface Interpretation Graphics
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Significant Updates

30/81
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Design Water Surface Elevation

- The top of barrier associated with the Design Water Surface Elevation plus superiority plus over-wash height (or wave runup overbuild) is termed the project (final) levee grade. Where superiority and over-wash height are zero, the final levee grade equals the DWSE.

- All standard geotechnical analyses (such as seepage, stability, and erosion) shall use the DWSE with the standard design requirements (such as effective stress/vertical gradient factor of safety, slope stability factor of safety, etc.).
Standards

- “in this engineering manual (EM), traditional design standards remain largely unchanged from previous editions, but a risk-informed process for evaluating required levee reliability for requesting variances from the standards when reliability is either too high (too expensive) or too low (too much risk).”
For existing levees, all future evaluations will be based on the same above risk-informed reliability evaluation process regardless of the method for how it was designed in the past. If the reliability of the existing levee is too low for the current estimated consequences, then the levee becomes a candidate for reevaluation within the USACE levee safety program routine processes, leading to further risk assessments and potential follow-on feasibility studies, congressional funding authorizations, and improvement design and construction.
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# ASTM Standard Tests

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<th>Test</th>
<th>ASTM Standard</th>
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<tr>
<td><strong>Soil Classification tests:</strong></td>
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<tr>
<td>Unified Soil Classification (USCS)</td>
<td>D2487 (testing/laboratory identification) and D2488 (visual/field identification)</td>
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<tr>
<td>Water Content</td>
<td>D2216 (preferred) D4648 (microwave method)</td>
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<td>Grain Size</td>
<td>D422 (coarse grain materials) D1140 (fine grain materials)</td>
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<td>Atterberg Limits</td>
<td>D4318</td>
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<tr>
<td>Specific Gravity</td>
<td>D854</td>
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<td>Organic Content</td>
<td>D2974 (Method C)</td>
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<td><strong>Soil Consolidation tests:</strong></td>
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<tr>
<td>Incremental Load Method</td>
<td>D2435</td>
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<td>Constant Rate of Strain</td>
<td>D4186</td>
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<td><strong>Soil Compaction test:</strong></td>
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<td>Standard Proctor</td>
<td>D698</td>
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<td><strong>Soil Shear Strength tests:</strong></td>
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<tr>
<td>Unconfined Compression</td>
<td>D2166</td>
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<tr>
<td>Triaxial Compression</td>
<td>D2850 (Unconsolidated, Undrained) D4767 (Consolidated, Undrained)</td>
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<tr>
<td>Direct Shear</td>
<td>D3080</td>
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<tr>
<td>Laboratory Miniature Vane Shear</td>
<td>D4648</td>
</tr>
<tr>
<td>Fully Soften Shear Strength</td>
<td>D7608</td>
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Geomorphology and Geophysics

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<th>Embankment deficiency or exploration target</th>
<th>Geophysical techniques to consider</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| Voids around metal (CMP) conduits          | 1. Temperature, Infra-red thermography  
2. Impact Echo (IE); other acoustic measurements | 1. Air-filled or water-filled voids may show anomalous temperature readings vs. sound backfill condition. Thermograms (thermal images) are taken from inside the conduit.  
2. IE frequency changes may indicate voids. Requires access to inside of conduit. |
| Locations of 'lost' or concealed metallic pipes | 1. Ground penetrating radar (GPR)  
2. Magnetometer / Pipe locator | 1. GPR penetration is site-dependent, may be limited in clay soils. Metallic pipes are readily detected by GPR beneath concrete slabs.  
2. Iron or steel pipe within a few feet of the surface can often be located by 'pipe locators' or magnetometers. Energizing pipe at one end generally improves detection by pipe locator. |
| Gradation changes along levees, changes in core/sheer configurations | 1. Resistivity / EM profiling  
2. Seismic surface wave; seismic refraction tomography | 1. Resistivity changes may indicate change from clays to silts, sands, etc.  
2. Seismic surface wave or refraction tomography may indicate core/sheer changes or gradation changes. |
| Animal burrows and associated voids within levees or embankments | 1. Electrical resistivity profiling  
2. Ground penetrating radar  
3. Seismic refraction tomography profiling | 1. ER profiling may indicate presence of air- or water-filled burrows. Electrode spacing may need to be tight, as this is a site-dependent survey.  
2. GPR imaging more effective in sands/silts; less effective in clays. Air-filled burrows easier to image than water-filled burrows.  
3. Refraction tomography profiling along the crest may indicate presence of burrows/other voids. Site dependent similar to 1. above. |
| Embankment fracturing, including desiccation cracking, differential settlement, subsidence | 1. Seismic profiling  
2. Resistivity profiling | 1. Seismic shear-wave profiling generally is sensitive to locations of transverse cracking. High-resolution reflection may indicate offsets related to settlement or subsidence.  
2. Resistivity profiling may indicate locations of air-filled transverse cracking. |
Standardized Material Graphics and Colors

Figure 6-1. Graphical and color standards for boring logs.
Guidance On Interpretation Process and Portrayal of Supporting Data
Interpreted and Analyzed Sections
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Analysis Methods

- Focus on modern FEM computer methods while still considering “Blanket Theory”
Effective Stress Heave/Uplift Design Criteria

The effective stress / vertical critical gradient factor of safety ($FS_g$) is computed assuming steady state seepage conditions for evaluation and/or design. The minimum $FS_g$ is based on the excess head at the bottom of the blanket directly beneath the levee toe. Variations in blanket thickness can be accounted for in Finite Element Method analyses or Blanket Theory computations, and the minimum $FS_g$ may be assessed beyond the levee toe. $FS_g$ is defined as:

$$FS_g = \frac{\gamma' \times z_i}{\gamma_w \times h_o}, \text{ which is the same as } FS_g = \frac{i_{cr}}{i_e}$$

where:

$FS_g$ = factor of safety based on effective stress / vertical critical gradient  
$\gamma'$ = average effective (or buoyant) unit weight of top stratum soil = $\gamma_{sat} - \gamma_w$  
$\gamma_w$ = unit weight of water  
$\gamma_{sat}$ = total, or saturated, unit weight of top stratum blanket  
$z_i$ = landside top stratum thickness (transformed thickness for blanket theory application or actual distance from the ground surface to the bottom of the least pervious strata when evaluating heads at that layer using FEM or FD techniques)  
$h_o$ = excess head (above hydrostatic) at the bottom of the blanket beneath the levee toe  
$\gamma' \times z_i$ = the effective stress at a point, typically at the base of the blanket  
$\gamma_w \times h_o$ = the excess pore pressure at a point, typically at the base of the blanket  
$i_{cr}$ = critical exit gradient = $\gamma' / \gamma_w$  
$i_e$ = vertical gradient at levee toe = $h_o / z_i$
Seepage Berm Design

- Use existing factor of safety criteria for critical vertical gradient / effective stress factor of safety for uplift at end of berms.
- If end of berm FoS is less than 1.6, then also assess likelihood of progression of backwards erosion.
Backwards Erosion / Piping Failure
Creep Ratios and/or Critical Horizontal Gradients

Bligh’s Creep Ratio and Lane’s Weighted Creep Ratio are empirical methods based on observations of seepage performance for a range of soil types. An evaluation of creep ratio is informative where either no blanket exists or there is a defect in the confining layer, providing some indication of the likelihood of backward progression of internal erosion leading to breach of the levee. Use of creep ratio to assess the need for berms where no blanket exists is described in EM 1110-2-1901 Paragraph 9-6b. TM 3-424 recommends adding seepage control measures where no landside blanket exists where creep ratios are lower than threshold values and flow is greater than 2 gallons per minute per foot (gpm/ft) of levee. While informative, this simple method is considered a “quick-and-dirty” check rather than a rational method of analysis by many in the profession (Duncan et al. 2011). Along the middle Mississippi River where very fine, uniform sands overlie a deep aquifer of coarser and more pervious sand, boils have rapidly progressed to failure where in-situ creep ratios are much higher than critical threshold values, which might have been misconstrued to have low probability of development of a backward erosion piping failure.

More recently, methods based on an average horizontal gradient have gained more attention for evaluating backward erosion of cohesionless soils, specifically based on the research of Schmertmann (1999) and Sellmeijer (1988). The research performed by both Schmertmann and Sellmeijer involved sand flumes in the lab to study the gradient across the structure required to achieve a complete breach. Full scale field tests (Van Beek et al. 2010) confirmed the retrograde erosion postulated by Terzaghi and later studied by Schmertmann and Sellmeijer. The renewed interest in these methods is helping to bring forward these experimental observations and incorporate previously acknowledged, but not widely applied considerations, such as gradational coefficients of uniformity. Consideration of an average gradient in cases where seepage flow is concentrated through a defect in an otherwise intact blanket may not be directly comparable with these empirical methods. A FEM analysis including the defect to assess local gradients in the vicinity of the unfiltered seepage exit may be more appropriate than simple average gradient computations.
Example With Vertical and Horizontal Gradient Considerations

Seepage Path Length No Berm
Creep Ratio Too Low

Seepage Path Length With Berm
Creep Ratio Sufficient (?)

Landside Berm
Meets USACE Vertical Gradient/
Effective Stress Design Criteria
& Increases Seepage Path Length

Clay Blanket
Sand Aquifer

Thin Blanket Does Not Meet
USACE Vertical Gradient/
Effective Stress Design Criteria
Vertical Boils Expected Here
Failure Modes & Event Trees

FAILURE EVENT TREE

INTERNAL EROSION DUE TO LEVEE UNDERSEEPAGE

Flood Fighting Impacts

Traditional Creep Ratio or Modern Horizontal Gradient Computations

Traditional Vertical Gradient / Effective Stress Computations
Filtered Toe Drain (aka Trench) Design

Very fine sand, $k_f = k_v = 3 \times 10^{-4} \text{ cm/s}$

Sand aquifer, $k_f = 1000 \times 10^{-4} \text{ cm/s}$, $k_v = 250 \times 10^{-4} \text{ cm/s}$

Waterside boundary applied to top of very fine sand is 25 ft higher than landside boundary applied to top of drain. All other external boundaries are no flow.

Figure G-3. Profile of FEM Analysis to Replicate TM 3-424 Example.
Figure H-1. General Plan View Flow Net of a Fully Penetrating Infinite Well Line and a Fully Penetrating Slot (Note flow is from a line source located a distance L from the well or slot).
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“Critical State” Soil Behavior Framework

- Critical State
- Normally Consolidated (NC)
- Over Consolidated (dense)
- Effective pressure
- Pore Water Pressure From Shear
  - Drained, “Slow”, Long-term
  - Undrained, “Fast”, Short-term
  - e Void Ratio

Shear and Volumetric Deformation

- Effective pressure - \( \sigma' \)
- Shear stress - \( \tau \)

- Peak (dense)
- Drained Fully Softened (NC)
- Residual (dense)
- Contraction
- Dilation or Swelling
- Grain Reorientation

- Pore Water Pressure or “Suction”
- No Change Water Can’t Get Out or In
- e Void Ratio

Strain
Summary of Types of Clays, Relative Strengths, Loading Conditions, Conventional Names and Conditions Controlled

- **Soft Clays**
  - Above the Critical State Line
  - OCR < say 2-4
  - "Wet of Critical"

- **Stiff / Compacted Clays**
  - Below the Critical State Line
  - OCR > say 2-4
  - "Dry of Critical"


**Drainage Condition**: Drained | Undrained Normally Consolidated Strength | Drained Fully Softened Strength - FSS | Undrained Peak &/or Undrained Strength

**Common Name**: Drained | Undrained | Drained | Undrained

**Condition Controlled**: Not Controlling In Nature, But can be measured In Lab | Controlling End of Construction & Rapid Flood Loading | Controlling Long-term Steady State Seepage OC Materials | Controlling During Transient Condition?
Selection of Strengths

\[ t_{99} = 4 \left( \frac{D^2}{c_u} \right) \]

Time for 99% Drainage

**Undrained Strengths**

- Dense, very over-consolidated
- High undrained strength
- Same soil and same initial pre-shear effective consolidation pressure
- Loose, under-, to normally to slightly over-consolidated
- Low undrained strength
- NC “strength”

**Drained Strengths**

- Dense, very over-consolidated
- High Peak Drained Strength
- Post Peak Strain-Softening
- Same soil and same initial pre-shear and sheared effective stresses

- Loose, under-, to normally to slightly over-consolidated
- Lower and No True Peak Drained Strength
- Always Strain-Hardening

- Peak Strain-Hardening
- Residual (when applicable)

- Fully Softened and Post-Peak

- Dense and NC “strength”
<table>
<thead>
<tr>
<th>Analysis Condition</th>
<th>Shear Strengtha</th>
<th>Pore Water Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case I. During Construction and End-of-Construction</td>
<td>Free draining soils - use drained strengths</td>
<td>Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady seepage analysis techniques (flow nets, finite element/difference analyses). Low permeability soils wet of critical – use total stresses with pore water pressures set to zero in the slope stability computations for materials with OCR &lt; say 2 to 4. Low permeability soils dry of critical – use effective stresses with appropriate construction pore pressures, often assumed to be hydrostatic-4.</td>
</tr>
<tr>
<td></td>
<td>Low permeability soils wet of critical – use undrained strengths based on pre-construction effective stress conditions for soils with OCR &lt; 2 to 4.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Low permeability soils dry of critical – use drained strengths when OCR &gt; 2 to 4.</td>
<td></td>
</tr>
<tr>
<td>Case II. Sudden Drawdown Conditions</td>
<td>Free draining soils - use drained strengths</td>
<td>Free draining soils - First stage computations (before drawdown) - steady state seepage pore pressures as described for steady state seepage condition. Second and third stage computations (after drawdown) - pore water pressures estimated using same techniques as for steady seepage, except with lowered water levels. Low permeability soils - First stage computations – steady state seepage pore pressures as described for steady state seepage condition. Second stage computations - Total stresses are used, pore water pressures are set to zero. Third stage computations - Use same pore pressures as free draining soils if drained strengths are being used; where undrained total stress strengths are used, pore water pressures have no effect and can be set to zero.</td>
</tr>
<tr>
<td></td>
<td>Low permeability soils - Three stage computations: First stage use effective stresses; second stage use undrained shear strengths and total stresses; third stage use drained strengths (effective stresses) or undrained strengths (total stresses) depending on which strength is lower - this will vary along the assumed shear surface.</td>
<td></td>
</tr>
<tr>
<td>Analysis Condition</td>
<td>Shear Strength&lt;sup&gt;a&lt;/sup&gt;</td>
<td>Pore Water Pressure</td>
</tr>
<tr>
<td>--------------------</td>
<td>-----------------------------</td>
<td>---------------------</td>
</tr>
<tr>
<td>Case III. Flood Loading</td>
<td>Free draining soils - use drained strengths. Residual strengths should be used where previous shear deformation or sliding has occurred. Low permeability soils wet of critical – use undrained strengths based on pre-flood effective stress conditions for soils with OCR &lt; 2 to 4. Low permeability soils dry of critical – use drained strengths using steady state seepage pore pressures.</td>
<td>Free draining soils - Pore water pressures can be estimated using analytical techniques such as hydrostatic pressure computations for no flow or steady seepage analysis techniques (flow nets, finite element/difference analyses). Low permeability soils wet of critical – use total stresses with pore water pressures set to zero in the slope stability computations for materials with OCR &lt; say 2 to 4. Low permeability soils dry of critical – use effective stresses under steady state seepage flood loading for materials with OCR &gt; say 2 to 4.</td>
</tr>
<tr>
<td>Case IV. Seismic</td>
<td>(see ETL 1110-2-XXX, Guidelines for Seismic Evaluation of Levees)</td>
<td></td>
</tr>
</tbody>
</table>

<sup>a</sup> Effective stress parameters can be obtained from consolidated-drained (CD) tests (either direct shear or triaxial) or consolidated-undrained (CU) triaxial tests on saturated specimens with pore water pressure measurements. Direct shear or Bromhead ring shear tests should be used to measure residual strengths. Undrained strengths can be obtained from unconsolidated-undrained (UU) and direct simple shear tests. Undrained shear strengths can also be estimated using consolidated-undrained (CU) tests on specimens consolidated to appropriate stress conditions representative of field conditions, but these strengths may be unconservative. The CU or “total stress” envelope, with associated c and φ parameters, should not be used. OCR is estimated based on the maximum past pressure and the effective stress prior to the flood load.

<sup>b</sup> For saturated soils with OCR < 2 to 4, use φ = 0.
For more information see 2013 ASDSO Conference Proceedings

SOME UNEXPECTED “MODERN” COMPLICATIONS IN SEEPAGE AND SLOPE STABILITY ANALYSIS: MODELING PORE PRESSURES AND STRENGTHS FOR FLOOD-LOADED STRUCTURES

Scott Shewbridge, PhD, PE, GE, CFM and Jeffrey Schaefer, PhD, PE, PG

![Graph showing shear strength and normal stress with labels for stiff, soft, dry of critical, and wet of critical.]

- Current EM
- Old EM
- Recommendation
- Normally Consolidated
Levee Seismic Risk
Post Seismic Warning / Response Time Scenarios

- Leveed Area
- Elevation
  - No Loss of Life
  - Reduced Damage
  - High Loss of Life / High Damage

Time

WSE

Earthquake

Long (evacuations and repairs)

None
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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:

\[ \dot{\varepsilon} = (k(\tau - \tau_c)) \]  

where

- \( \dot{\varepsilon} \) = erosion rate
- \( k \) = erodibility coefficient or detachment rate coefficient (ft³/lb-hr)
- \( \tau \) = effective hydraulic stress on the soil boundary (lb/ft²)
- \( \tau_c \) = critical shear stress (lb/ft²), i.e. the shear stress at which erosion starts

10.3.2 The erosion rate (\( \dot{\varepsilon} \)) is a function of both hydraulic (\( \tau \)) and geotechnical (\( k, \tau_c \)) parameters. Effective hydraulic stress (\( \tau \)) mainly depends on characteristics of water-soil boundary, current/stream velocity and/or wind wave height and period. Both \( k \) and \( \tau_c \) 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.
Typical Analysis Parameters
Design of Overtopping - Resiliency Measures

Initial Stage A: Erosion due to overtopping on the Citrus Bach 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
Design of Overtopping - Resiliency Measures

Table 10-8
Erosion Rates for Base Clay (CSU Study).

<table>
<thead>
<tr>
<th>Bare Clay</th>
<th>$q = 0.1 \text{ cfs/ft}$</th>
<th>$q = 0.2 \text{ cfs/ft}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3:1 (H:V) Slope</td>
<td>1 ft/hr</td>
<td>2 ft/hr</td>
</tr>
<tr>
<td>25:1 (H:V) Slope</td>
<td>0.5 ft/hr</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 10-9
Max. sustained flow of landslide armoring materials
(From Table 10-7, LARDR October 2013)

<table>
<thead>
<tr>
<th>Max. Time-Averaged Wave Overtopping Discharge (cfs/ft)</th>
<th>Armoring Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0</td>
<td>Dormant Bermuda Grass*</td>
</tr>
<tr>
<td>4.0</td>
<td>HPTRM Reinforced dormant Bermuda Grass and ACB</td>
</tr>
</tbody>
</table>

* The quality of the Bermuda Grass cover may be supplemented with a program of fertilizer and water application if the designer has concerns regarding the ability to meet root volume requirements (see LARDR, October 2013).
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.
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USBR / USACE “Best Practices”
Failure Modes & Event Trees

Flood Fighting Impacts

Traditional Creep Ratio or Modern Horizontal Gradient Computations

Traditional Vertical Gradient / Effective Stress Computations
Risk-Informed Design Progression

- Low Population Levee
- High Population Levee
- Existing Conditions
- Minimum Berm
- Minimum Berm + Flood Fighting
- Higher Standard Berm
- Higher Standard Berm + Flood Fighting
- Higher Standard Berm + Flood Fighting + Improved Emergency Evacuation

Graph shows the relationship between Life Loss and Annualized Probability of Failure, with TRRL indicated on the graph.
How “Reliable” Does the Levee Need to Be For Prior-to-Overtopping Failure Modes When Overtopping Controls?
Taylor Series Approximate Solution – First Order Second Moment
Can be used with virtually any limit state analysis.

Only Models Aleatory

- Vary each factor by +/- 1 sigma,
- Look at variation in response function,
- Use that as estimate of variance of response function

- Compare distance of “Limit State” to “Expected” value and based on variance estimated above, calculate percentage of area under lognormal curve for values greater than the limit value.
Epistemic Uncertainty
“Unknown-Unknowns”

For geotechnical engineering, epistemic uncertainty (knowledge uncertainty) in many situations is more important than aleatory (natural variability), because it often cannot be estimated directly and can have dramatic impact.

For geotechnical failure modes, both for design and reliability assessments, the epistemic uncertainty “unknown-unknown” challenge is common and well-known to the profession and has been addressed through a classic inductive-reasoning approach referred to as the Terzaghi and Peck “Observational Method” (Peck 1969).
Epistemic Uncertainty Risk Mitigation and Levee Safety Program Non-Routine Activities

Recognizing that it is not possible to eliminate all “unknown-unknowns” on levee systems, per current and past USACE practice, performance monitoring and flood fighting will remain important risk reduction measures to reduce epistemic uncertainty over time. All failure mode event tree analyses will include event nodes for Unsuccessful Detection and Unsuccessful Intervention.

As a part of the proposed levee safety program (Draft EC 1110-2-6072, USACE, 2014), if performance of a levee is deemed too poor and flood fighting activities too demanding and unreliable, a risk-informed process will be initiated to evaluate the needs for additional actions. If risk is high enough, a variety of structural and non-structural actions to increase levee reliability (i.e., change the fragility curves) and/or decrease potential consequences will be considered.
Failure Modes & Event Trees

Diagram showing the sequence of events leading to a breach, including detection, need for intervention, and success of intervention.
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14.16 Flood Performance and Special Considerations. The PDT shall identify the expected normal performance of the project during flood events, including the expectation for “normal” design performance that requires “flood fighting” activities. The designer should give indications of various measures of performance, such as seepage flow rates, and the magnitude and extent of flood fighting activities (for example, localized pin boils and small sand boils distributed broadly along entire length of levee system, with larger boils likely between river mile x and y). If instrumentation is in place, the PDT shall identify threshold levels with corresponding actions required by the Local Sponsor. Any critical area, such as seepage concerns, should be identified to the Local Sponsor.

14.17 Flood Fighting Expectations.

14.17.1 In some regions, flood fighting is considered a normal expectation and is included as an expectation during the development levee design, especially for water levels approaching the design water level. For example, truncated seepage berms with heave/uplift factors of safety of 1 at the berm toe are expected to produce seepage boils, requiring increased monitoring, sand bagging, and other flood fighting activities to prevent progression of backwards erosion failure of the berm and levee. The Local Sponsor shall increase surveillance of the levee when the flood water elevation is above the ground surface elevation at the dryside/protected side toe of the levee and undertake appropriate flood fighting activities, as required.
Example Flood Fighting Evaluation
“More than expected and, but for flood fighting, levee would have failed”
Ensley Berm, Memphis 2011

Levee did not fail, but internal erosion pipes projecting towards the river found in 2012.
Example Flood Fighting Evaluation
“Flood fighting occurred but levee failed”
L-575 Breach, NW Atchison County Levee District, Hamburg Iowa 2011

Possibly due to defects in riverside cap - fourth pipe formed and breached on June 13, 2011.
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Schedule

- Independent External Peer Review (IEPR)
  - Contract in Negotiation
  - Anticipated Completion December 2015
- More Review?
- Publication FY2016 Q3 or Q4
Questions / Discussion