WEST Sacramento Levee Improvement Program

Board of Senior Consultants

Report on BOSC meeting of March 26-27, 2013
WSAFCA Southport Project

Report Prepared by:

Board of Senior Consultants:

Dr. David T. Williams
Dr. Ray E. Martin
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April 29, 2013

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Mr. Ken Ruzich
General Manager
West Sacramento Area Flood Control Agency (WSAFCA)
1110 W. Capitol Ave.
Sacramento, CA 95691

Subject: Report on BOSC meeting of March 26-27, 2013, Southport Project, West Sacramento, CA

Dear Mr. Ruzich:

This report was prepared by the Board of Senior Consultants (BOSC) for the purpose referenced above. The BOSC does not consider this report to be the end of the 65% review process by the BOSC. The BOSC meeting on March 26 – 27, 2013 provided an excellent opportunity to discuss many items with the design team. The BOSC wishes to have a continued dialogue with the design team on the items in the 65% plans and specification as well as those items in this report as the project moves forward to 90% plans a specifications. Please note that the BOSC opinions on many of these issues are expressed in Appendix C: Instructions to the Board.

We appreciate the effort that went into the required coordination of the groups for this meeting and believe this to be best organized meeting the BOSC has attended. Thanks to all for organization of the meeting – went well and smooth.

The following are the BOSC comments on items that were presented at the meeting as well as related material that had been previously sent to the BOSC.

Riprap

1. Riprap placed over existing riprap. In areas where slopes have existing riprap protection and this protection has been in place for a long duration, the design team should consider placing additional riprap on top of the existing riprap without application of bedding material or other filter considerations if there is a high degree of confidence that existing riprap does not have fines piping through the riprap. Engineering judgment could be used to justify this application. This will reduce unnecessary costs.

2. If putting the riprap underwater can be done very accurately by such instrumentation as side-scan sonar, the 1.50 multiplier for riprap thickness can be reduced per EM 1110-2-1601.

3. Revetment depth means thickness perpendicular to layer orientation, not the vertical distance as shown in some of the slides.

4. Reexamine the factors of safety being applied to the design of riprap protection of non-flood protection features.

Erosion Control Sites

5. In the ranking of the erosion control sites where riprap criteria is not met, it should be noted if the riprap is still performing and protecting the bank and the score should be adjusted accordingly. Additionally, the design team should consider adding another column in its ratings that uses engineering judgment to evaluate the historic performance of the existing riprap.

6. Also, riprap should be tabulated to indicate if its placement is needed to protect the levee or designed to hold the existing bank alignment or remnant levee in place.

7. If there is mitigation credit for placing “environmentally friendly” protection such as vegetated riprap (that may qualify as habitat), these should be taken into consideration in the erosion sites prioritization/ranking.

8. The erosion site priority should include a category that takes into consideration if a site helps complete a contiguous segment of erosion control. If the site accomplishes this, then it should have a higher ranking.

9. For the erosion risk categories, there should be a scale of 1 – 10 and the current variable scales should be adjusted accordingly to reduce confusion. The scoring and weighting steps should be segregated rather than combined into one numerical ranking system.

10. The scoring and weightings determinations for determining the erosion site priority should include the WSAFCA staff as well as other interested parties, not just the design team.

11. USACE has had 50 years of erosion projects in this area and all of them have failed over time. The reasons and modes for these failures should be evaluated in relation to the proposed design.

Offset Areas

12. Consideration should be given to using some of the higher LL material (above 65-70) to line the inlet and outlet areas in an effort to reduce project cost.

13. For the riprap design of the inlet and outlets of the offset areas, what n values were used (initial, intermediate, and mature)?

14. The designers should consider having the offset openings just at the downstream and none at the upstream (or a smaller inlet) so it floods by backwater into the overbank area at a level pool. This would greatly decrease the need for riprap protection for the offset entrances since it would not be a flow through condition.

15. For the offset area, consideration should be given to having a peaked stone weir for the floodplain apron that can be adjusted to regulate the frequency of offset inundation. This approach will provide flexibility for establishing inundation frequency and support adaptive management practices.

16. If the design team selects a flow-through design, consideration should be given to creating a situation so that when the water enters from the upstream inlet, water has already entered the offset from the downstream opening. This could decrease the riprap requirements of the offset entrances.

17. Consideration should be given to using "fat" clays to seal the offset inlet and outlet locations to reduce potential erosion.

18. The design team should consider more specific information on water residence time and frequency of inundation. The offset areas and adaptive management strategies should then be designed accordingly.

19. For erosion control measures of the remnant levee and inlet and outlet locations of the offset areas, WSAFCA should consider the tradeoffs between capital improvements and adopting an adaptive management plan that has O&M implications.

Hydraulic models

20. Slide - Hydraulic Analysis Assumption in the ULDC: The overflow basins generally have large enough storage capacity to where water does not flow back into the system. However, if the assumptions is not to fail the downstream levees containing the overbank storage and prevent the water from flowing back into the system, it is a non-conservative assumption.

21. MBK should work with the USACE to secure a written agreement on any proposed refinements to the HEC-RAS Model release 2.

22. The BOSC suggested that if the 2D model WSELs are consistently lower than the 1D model for the 200-year flood, the design team may not need to do analysis with the 2D model for the 500-year event.

23. The BOSC notes that USACE regulates the design to the 1957 profile so WSAFCA's design WSEL is based on the ULDC and is meant to meet those criteria. To ensure robust design, WSAFCA used a 200-yr WSEL plus 1 foot for geotechnical design to account for the potential of future variability and changes to the WSELs from new hydrology, climate change or revisions to hydraulic models. This approach is endorsed by the BOSC and adds to the Redundancy, Robustness and Resiliency (3 Rs) of the project.

24. The 65% submittal assumes a mature vegetation condition. The design team should evaluate initial and intermediate vegetation maturity levels to determine the appropriate hydraulic design conditions.

Bees Lake

25. There was considerable discussion about how to analyze the potential for a failure of the existing levee following completion of the new levee west of Bees Lake and what impact a failure would have on the new levee. It was implied by the design team that this was a breach issue and a breach failure analysis was proposed. This is an existing seepage problem and may or may not become a breach problem. There is known seepage to the area as the lake has historically

fluctuated somewhat with the river level. The fact that there is seepage occurring does not mean that piping has, or is, or will occur. This should be analyzed as a seepage problem first. The head difference between Bees Lake and the river that has existed in the past under flooding conditions should be obtained from historical data. The maximum river stage and lake level should be used in a seepage analysis to estimate the gradients. The Bees Lake cross section on Exhibit A-3.5.7.47 indicates the levee was constructed of SP-SM material but Boring WR0900_059B, which is not shown on the cross section, indicates it is SM. This is underlain by an ML layer at El 12 to -6 and then SP, SP-SM material below EL -6. The bottom of the lake is shown as EL -6. It is likely that the seepage connection is below the ML layer, otherwise seepage would be exiting from the toe of the levee and we are not aware that that has occurred. If underseepage analysis indicates the gradients suggest piping should be occurring, the analysis is likely conservative. Piping is progressive and it is likely that the condition has existed long enough that if failure was going to occur, it would have by now. Once a reasonable gradient has been estimated, the analysis should be repeated using the design head and the same lake level used previously. This will be conservative as a higher river head will induce more flow and a higher lake level. If this analysis indicates gradients are too high, then methods to mitigate the problem should be given consideration.

26. If a surge analysis is performed and it shows potential problems and the worst case for breaching of the Bees Lake area is from the upstream road embankment, consider reinforcing it. Therefore, if there is failure, it is at the downstream area, resulting in flooding from backwater, not from an upstream surge.

27. Breach analysis may be too conservative and should consider that some water would exist in Bee's Lake.

<u>Fill Material and Fill Area</u>

28. For fill design, combine Type I and Type 1A and describe them as the same as the designers have for 1A. Also, Type 1 should not have SM since if PI is greater than 8, it is SC – but does not matter if type 1 and 1A is combined.

29. BCI should consider developing quantity estimates in the Yarbrough borrow site based on the liquid limit available material. As existing explorations move west in Yarbrough site, the material becomes a "fatter" clay.

30. The BOSC agrees that a factor of safety of 2 is reasonable for borrow demand.

<u>General</u>

31. When the designers are laying out tree plantings for the riverside excavated flow areas, they should consider <u>not</u> planting any cottonwoods along the riverside berm areas next to the levee. Cottonwoods have extensive root systems that have been documented to extend many feet beyond the tree and penetrating slurry cutoff walls in the Central Valley.

32. A literature review has been developed and attached to this document (Appendix D) with recommendations concerning how to mitigate desiccation cracking of the CH clays without encapsulating them with 10 feet of CL material. We ask for careful consideration of these

recommendations by both the design team and the Corps. Of particular interest related to this topic is IPET Team, Sills, G. L. (major contributor), et. al., "Interagency Performance Evaluation Taskforce (IPET), (2006) and "Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, Vol. V," U.S. Army Corps of Engineers, Draft Final Report. In addition, Vicksburg District has developed a report that states if clay is covered by 3 feet of material, they do not have cracking issues. SBFCA is using 18 inches of material over the levee shell.

33. The change in local drainage from pre and post project conditions should be explicitly determined and presented.

34. Slide Segment B2 Section: The design team needs to show how settlement of the cutoff wall does not cause a window in the cutoff wall. The cutoff wall should be built on a working platform that is at least 3 to 5 feet thick.

35. Slide Segment B2 Section: Final under-seepage should be analyzed using the proposed 40 foot wide waterside berm.

36. Segment E Strip Map: The design team should work with the State and the USACE to determine long term O&M responsibilities for the project.

37. Segment G Section: The design team should consider a reduced amount of degrade of the existing levee and evaluate if a clam shell could be used to complete the cutoff wall. The Specifications already require that the contractor must have a clam shell on site to terminate some of the slurry walls into a clay layer. The team should consider performing additional CPTs and see if some of the deeper walls could be shallower and still tie into an adequate cutoff wall finish layer. This could provide some cost savings from a reduction n the amount of levee degrade and borrow required to complete the project.

38. The BOSC would like the design team to revisit the underseepage model assembled for Segment E to determine if underseepage measures are necessary.

39. The BOSC suggests cutting the number of materials to 2 types to save money on construction. HDR noted that three material types may be needed: one for the shell and two for the core with differing liquid limits. A 2 type section is preferable and will likely result in significant construction cost savings.

40. The design team should characterize the potential for erosion on the waterside berm.

41. WSAFCA should work with the USACE to share the results of the side scan sonar results.

42. The BOSC would like to see the modified rapid drawdown scenario for the waterside bench of the setback levee.

43. The BOSC requests the results of one rapid drawdown analysis for the worst case scenario and provide the resulting factor of safety. The BOSC request to see which section is considered the worst case before the entire analysis is run.

44. The design team should describe the activities that will be required for O&M.

45. If access is limited, a foot patrol at the toe of the waterside berm could be performed. The design team should consider requiring this as an O&M procedure.

Very truly yours,

West Sacramento Levee Improvement Program
Board of Senior Consultants

Dr. David T. Williams, P.E., CFM, PH

Mr. George L. Sills, P.E.

Dr. Ray E. Martin, P.E.

<u>Appendices</u>

Appendix A: MEETING AGENDA

Appendix B: CHARGE TO THE BOARD

Appendix C: INSTRUCTIONS TO THE BOARD

Appendix D: Literature Review, Desiccation of Clay Soils

Appendix A

WEST SACRAMENTO AREA FLOOD CONTROL AGENCY
MEETING AGENDA

WEST SACRAMENTO LEVEE IMPROVEMENT PROGRAM
BOARD OF SENIOR CONSULTANTS
MEETING NO. 7

Date:	March, 26-27, 2013	
Time:	8:00 am to 5:00 pm	
Location:	West Sacramento City Council Chamber 1110 West Capitol Avenue West Sacramento, CA 95691	Parking 1020 West Capital Ave. West Sacramento, CA 95691

DAY 1

I. INTRODUCTION — **8:00 AM – 8:30 AM**
- Welcome and Opening Remarks (WSAFCA)
- Program Context and Background (WSAFCA)
- Meeting Purpose & Expectations (MBK)
- Agenda Overview (HDR)

II. OVERVIEW 65% DESIGN (HDR) — **8:30 AM - 9:30 AM**
- Design Team Members
- Levee Deficiencies
- Proposed Corrective Measures and Project Inventory
- Typical Levee Sections

BREAK — **9:30 AM – 9:45 AM**

III. HYDRAULIC BASIS OF DESIGN (MBK) — **9:45 AM – 10:30 AM**
- Model Description and Data Sources
- Model Calibration
- Design Water Surface Elevations

BREAK — **10:30 AM – 10:45 AM**

IV. GEOTECHNICAL BASIS OF DESIGN(BCI) (Part 1) — **10:45 AM – 11:45 AM**
- Design Criteria
- Segment A Basis of Design
- Segment B1 Basis of Design

LUNCH — **11:45 AM – 12:15 PM**

V. GEOTECHNICAL BASIS OF DESIGN (BCI) (Part 2)

(Catered in Community Center) **12:15 PM – 1:15 PM**

- Segment B2
- Segment C
- Segment D

BREAK **1:15 PM – 1:30 PM**

V. GEOTECHNICAL BASIS OF DESIGN (BCI) (Part 3) **1:30 PM – 2:15 PM**

- Segment E
- Segment F
- Segment G

BREAK **2:15 PM – 2:30 PM**

VI. GEOMORPHIC EVALUATIONS (CBEC) (Part 1) **2:30 PM – 3:30 PM**

- Field Survey Overview – Existing Rip Rap and Erosion
- Development of Erosion Control Basis of Design / Mike 21 Modeling

BREAK **3:30 PM – 3:45 PM**

VII. GEOMORPHIC EVALUATIONS (CBEC) (Part 2) **3:45 PM – 4:45 PM**

- Erosion Control Recommendations
    - Strengthen-in-place levee sections
    - Soft measures
    - Inlet/Outlets
    - Adaptive Management

VIII. WRAP-UP / DAY 2 AGENDA REVIEW **4:45 PM – 5:00 PM**

DAY 2

IX. GEOMORPHIC EVALUATIONS (CBEC) (Part 3) **8:00 AM – 9:00 PM**

- Erosion Control Design – continued discussion
- Upcoming Survey of Existing Channel Toe Rip Rap
- Breach Analyses for Bees Lake

BREAK **9:00 AM – 9:15 AM**

X. SITE CIVIL DESIGN (HDR) (Part 1) **9:15 AM – 10:15 AM**

- Borrow Inventory and Project Fill Demand
- Levee Zoning and Fill Specifications
- Offset Area Civil Design

BREAK **10:15 AM – 10:30 AM**

XI. SITE CIVIL DESIGN (HDR) (Part 2) **10:30 AM – 11:30 AM**
- Utility relocation
- Transportation
- Operation and Maintenance

XII. LUNCH/BOSC (CLOSED WORKING SESSION) **11:30 AM – 1:00 PM**
- Note: Design team to be available, as needed, to address BOSC questions

XIII. SUPPLEMENTAL GEOTECH EVALUATIONS (BCI) **1:00 PM – 2:00 PM**
- Marina access embankment alignments/settlement
- Marina access embankment fill specification
- Additional Borrow Evaluations

BREAK **2:00 PM – 2:15 PM**

XIV. OFFSET AREA MITIGATION DESIGN (ICF) **2:15 PM – 3:15 PM**
- Habitat Sustainability Criteria
- Habitat Design

BREAK **3:15 PM – 3:30 PM**

XV. REVIEW COMMENTS (CLOSED SESSION) **3:30 PM – 4:30 PM**
- Overview of Comments
- Comment Clarification & Discussion
- Summary of Actions for Comment Resolution

XVI. CONCLUSIONS & ACTIONS (CLOSED SESSION) **4:30 PM – 5:00 PM**

Appendix B

WEST SACRAMENTO LEVEE IMPROVEMENT PROGRAM
BOARD OF SENIOR CONSULTANTS

CHARGE TO THE BOARD

The West Sacramento Area Flood Control Agency (WSAFCA) has assembled this Board of Senior Consultants (Board) to conduct an independent and external expert review of the levee improvements under design by the WSAFCA and its consultants for construction. The Board is charged with confirming that the design investigation and analysis and associated recommendations for levee improvements at each site are acceptable for providing 200-year level of flood protection in an urban environment. The Board shall consider current and relevant regulations, policy, standards, and guidance for the design and construction of flood protection measures in rendering its opinion. The Board shall identify potential areas where cost savings could be achieved. The Board shall document its findings that will include, but is not limited to, responding to the instructions provided by WSAFCA. WSAFCA shall be responsible for providing the Board with instructions, the historic data and records, programmatic or planning studies, and design phase data and documentation necessary to understand the technical context and natural setting within which the levee improvement recommendation has been proposed.

Appendix C

WEST SACRAMENTO LEVEE IMPROVEMENT PROGRAM
BOARD OF SENIOR CONSULTANTS

INSTRUCTIONS TO THE BOARD

WSAFCA requests that the Board specifically consider the following concerns:

1. Are there any recommendations related to the development of adequate basis of design documentation?

   The BOSC comments on the 65% submittal by HDR are documented in the BOSC report entitled "BOSC Review Report on the 65% Design Documentation Report and Plans and Specifications by HDR for the WSAFCA Southport Project, West Sacramento, CA" dated March 20, 2013.

2. What are the soil parameters that should qualify as suitable embankment fill for construction?

   The design team should consider basing the soil parameters used in the design of the embankment fill for construction upon the following, which are founded upon the soil properties provided from the borrow areas.

   Materials testing

   Obtain soil material with LLs of about 45, 60, and 75.
   Determine optimum moisture contents for each sample.
   Compact the tri-axial samples at the optimum moisture contents, and
   Test the samples for strengths.

   Working platform

   Assuming that the soil strengths obtain from this procedure are adequate, the BOSC recommends that a 5 foot thick working platform be constructed out of soils that have LLs greater than or equal to 60 and less than or equal to 75. This soil material must pass the following condition: for an average of 10 consecutive tests, the LLs are less than or equal to 80 for any individual test.

   Levee Shell

   LLs less than or equal to 45, average 10 consecutive tests, less than 50 for any individual test.
   The PI must be less than 40 and fines greater than 12%.

Levee Core

Use material type 2 as defined by a LL greater than or equal to 35, less than or equal to 75. This soil material must pass the following condition: for an average of 10 consecutive tests, the LLs are less than or equal to 80 for any individual test. The PI must be greater than or equal to 8 and less than equal to 55.

3. Is the erosion site priority scoring system sufficient for defining why a site ranks as a low, medium or high priority site?

   The BOSC recommends segregating the rating such that each evaluation element has the same range (suggest 1 to 10) and a weighting factor assigned to each element. This would conform to traditional methods for prioritizing erosion sites.

4. Are the explorations conducted to date sufficient to evaluate the Southport EIP project from a geotechnical standpoint?

   The levee explorations are sufficient except for Segment G. For this Segment, the designer should consider additional explorations to better define the bottom of the proposed slurry wall. Then the designer could establish a fixed depth of the slurry wall and if justified, consider removal of the additional requirement to go deeper in this segment.

   The BOSC recommends additional explorations for the borrow areas based upon a grid system and more soil classification tests for the full depth.

5. Are the cross-sections evaluated for 65% design adequately representing critical locations along each Southport EIP Segment, with the understanding that an additional cross-section in the northern portion of Segment G will be developed for 90% design?

   The BOSC agrees that the cross section locations adequately represent critical locations for the 65% submittal.

6. Do the subsurface profiles adequately support the design and represent the subsurface conditions for each Southport EIP Segment?

   These subsurface profiles are currently under review so the BOSC reserves comments on this question.

In providing commentary on these and other matters related to the documents reviewed for these projects, please provide the following where possible:

- A clear statement of the degree of concern;
- The basis of the concern;
- The significance of the concern; and
- The actions needed to resolve the concern

Appendix D

West Sacramento Area Flood Control Agency
Southport Levee Improvement Project
Literature Review
Desiccation of Clay Soils
By
Board of Senior Consultants
April 29, 2013

Purpose

This literature review was performed to evaluate whether a thin soil cover layer could be used over highly plastic CH clay soils to protect the CH soils from: 1) desiccation cracking and resulting weathering, and 2) reduction to the fully softened strength with the subsequent potential for shallow maintenance type slope instability. This approach will also allow the use of more of the available highly plastic clay soils from the Yarbrough Borrow Area.

Introduction

The following is a brief summary of some of the available literature related to desiccation of clay soils. The various papers referenced were obtained: 1) by members of the Board of Senior Consultants (BOSC), 2) through the support of the Center for Geotechnical Practice and Research at Virginia Tech and particularly by Drs. Jim Mitchell and Tom Brandon; and 3) by discussions with Dr. Craig Benson at the University of Wisconsin-Madison. We could find no papers that describe this application specifically but several papers provided significant insight and proved valuable in developing the recommendations that are presented at the end of this literature review.

The literature review is separated into three sections. The first section provides review of ten references that present general background data on the subject of desiccation. The second section provides review of a research report concerning desiccation of waste disposal covers which has application to the issue under consideration. The third section covers issues related to stability of embankments constructed of clays which are subject to desiccation. A Summary section of all literature reviewed is followed by Conclusions and Recommendations. Referenced discussed and other references that were reviewed but not discussed or were referenced in the articles that were reviewed herein are listed at the end of this document in the Reference section.

Review of General References

Fundamentals of Soil Behavior (Mitchell (1993)

Mitchell indicated that the type and amount of clay mineral present in a drying soil controls desiccation cracking. Fine-grained soils have smaller pores and thus are more susceptible to the development of cracks than coarse-grained soils. This is caused by the development of higher suction due to the smaller pores. The presence of highly active clay particles in larger quantities promotes crack formation to a greater extent than soils with less active clay mineral and a lower percentage of clay size particles.

Desiccation Cracking of Soils (Kodikara, et al (2000))

The authors reviewed the work of others to determine required future research and reported the results of Corte and Higashi (1960) who found that the desiccation cracking water content decreased as the desiccation rate increased as shown in the Figure 2 below. The cracking water content is shown on the figure by the arrows. The desiccation rate was varied by control of temperature and humidity.

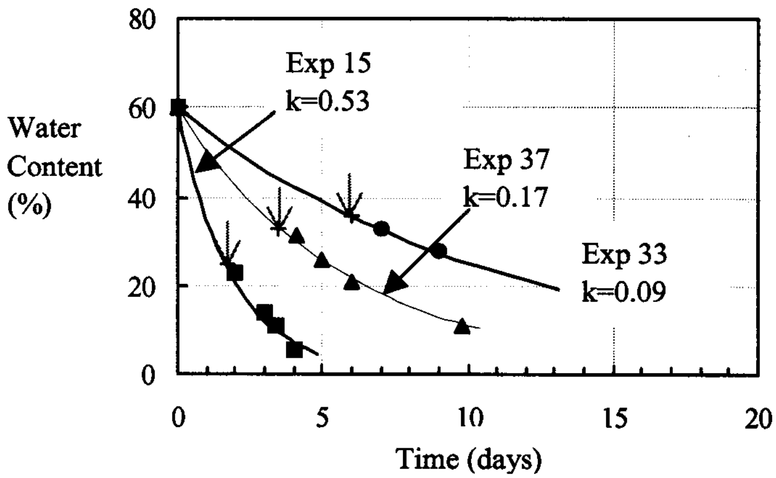

Figure2. Process of desiccation at the surface soil (Corte and Higashi, 1960).

Shrinkage and Cracking Behavior of Swelling Soil Under Different Temperatures (Tang (2007, 2009))

Evaporation begins on the soil surface and results in the development of tensile suction stresses. When these stresses exceed the bonding strength of the soil, desiccation cracking begins and this is called the cracking water content, w_c. Kayyal, et al. (1995) reported that the increase in suction stresses within a soil mass is related to temperature and humidity. After desiccation cracks begin to form, the surface crack ratio, R_{sc}, increases almost linearly with decreasing water content as shown by Figure 6 from the paper for a CH clay (LL=77, PI=37). R_{sc} is defined as the ratio of the surface area of cracks to the total surface area of a specimen. The relationship between the R_{sc} and water content, which was defined as the cracking curve, was found to stabilize as the water content reached the shrinkage limit, SL, of the soil. The cracking curve was found to reflect the shrinkage properties of the soil.

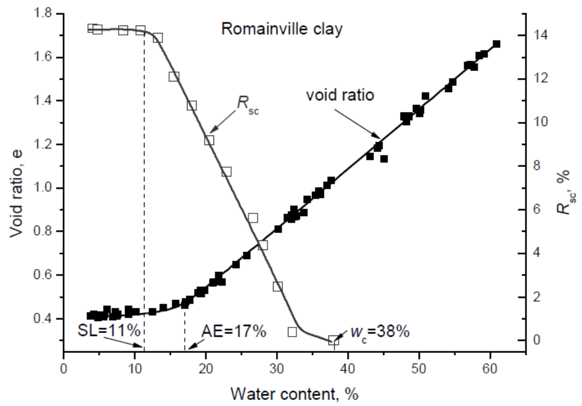

Figure 6. The changes of void ratio and R_{sc} with water content during drying (Romainville clay specimens)

The study found that as the temperature rises, the cracking water content, *w*c, rises and the residual water content at equilibrium declines. The water loss rate was also found to increase with temperature, e.g., the time to reach equilibrium water content also was reduced as shown in Figure 3 from the paper for a CL clay (LL=37, PI=17).

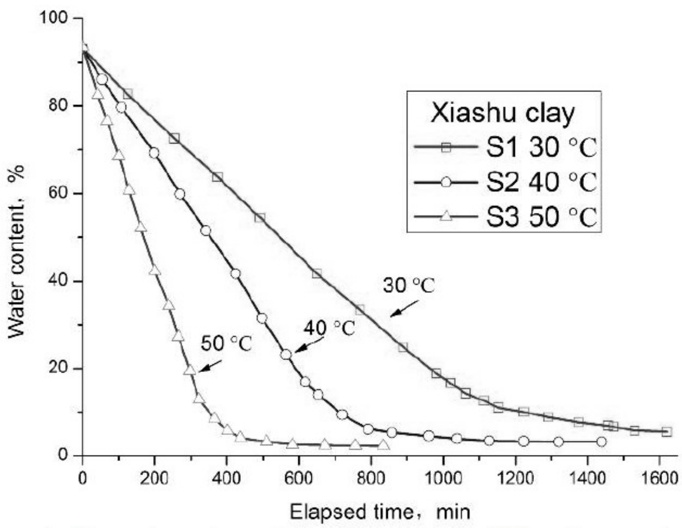

Figure 3. Change in water content with time under different temperatures.

Numerical Modeling of Desiccation Cracking in Compacted Soils (Inci (2008))

The report highlighted the factors that affect the shrinkage and desiccation cracking of soils including: clay mineralogy, clay content, compaction conditions, drying process, wetting and drying cycles, soil particle orientation, unit weight, and pore fluid.

Daniel (1991) found that shrinkage is related to plasticity index and is highest when the plasticity index is high and shrinkage limit is low. Daniel and Wu (1993) recommend that clayey sand be used for liner construction at arid sites because it combines lower hydraulic conductivity and lower shrinkage potential. They also recommend compaction at lower water contents with high relative compaction to reduce cracking in arid areas. Yong and Warkentin (1975) found that irreversible fabric changes occurred during the first drying cycle in a clay soil.

The author found that the Finite Element Method was an effective tool to model desiccation cracking and that the *"water content profile, and resulting stress and modulus variations, are more critical for cracking potential and crack propagation."* .

Cracks in Soils Related to Desiccation and Treatment (Taha, et al (2011))

The authors noted that the purpose of the paper was to *"review previous studies, explain the problems encountered, and then suggest a way forward for future studies."* They reported that Omidi (1993), Albrecht and Benson (2001), Osinubi and Eberemu (2010), Osinubi and Nwaiwu (2008), Harianto, Hayashi *et al*., (2008), and Puljan (2010) found that shrinkage strain depended on three main parameters: molding water content, compaction effort (dry density), and soil plasticity index. They concluded that shrinkage strain and thus desiccation cracking increased with increasing plasticity index and molding water content. They also concluded that increased dry density lead to a decrease in shrinkage strain and desiccation. Thus, as molding water content decreases and molding density increases, shrinkage of the soil decreases. They also noted that lime treatment of soils is not effective in reducing desiccation (Guney, et al. (2007)) but that cement treatment of clay soils with a PI <20 was effective (Walker (1995)) and that fiber treatment of silty soils was effective in reducing cracking (Rifai and Miller (2009)).

Lime Stabilization of Levee Slopes (Fleming, et al (1992))

The authors found that lime treatment of highly plastic clays (CH) was effective when repairing shallow surficial slides up to five to seven feet deep along Mississippi River levees in Mississippi and Louisiana because it ameliorated the properties of these clay. After treatment the LL and PI of these clays were reduced and the soils classified CL and ML. The levees along the Mississippi River were constructed on CL and CH clays. The Corps of Engineers performed a study of these levees and found that the slides only occurred where the levees were constructed of highly plastic clays classified CH and stated,

> *"---Atterberg limit indices of materials recovered directly from slides---indicate*[d] *that materials susceptible to slough slides may be characterized as having a liquid limit greater than 60 and a PI greater than 40."*

The data indicated that there were no slides in areas where the PI was less than 27 and *"very few where the PI is between 27 and 40."* No slopes constructed of clays classified CL experienced these shallow slope failures. As of the date of the publication 142 slides were repaired using lime treatment to ameliorate the highly plastic clays. Originally the entire slide depth was replaced with lime treated compacted clay. From 1990 forward only the outer 3ft of material was lime treated. A personal communication from Stewart (2013) indicated that no slides that were repaired with the 3ft thick lime treated layer have failed in the intervening 20+ years.

Desiccation Cracking Of Soils (Lau (1987))

The thesis describes two approaches to evaluating the volume change behavior of unsaturated clay soils: the first was derived using volume change behavior (elastic equilibrium analysis) and the second using shear strength behavior (plastic equilibrium analysis). Desiccation cracking is the result of volume reduction due to a change in matric suction. Matric suction is exerted by the soil matrix which induces water to flow from a soil with low matric suction (a wet soil) to soil with high matric suction (a dry soil). It is a negative pressure that results from the combined effects of adsorption and capillarity. Fredlund and Morgenstern (1977) defined two stress state variables that were found to be useful were $\sigma - u_a$ and $u_a - u_w$, where σ = total normal stress, u_a = pore air pressure, and u_w = pore water pressure. The first is termed the net total stress and second is matric suction. The term u_a was assumed to be zero. Lau stated:

> *"The crack depth predicted by the plastic equilibrium analysis is almost twice as deep as that predicted by the elastic equilibrium analysis. Since the formation of desiccation cracks is the result of soil volume reduction, the elastic equilibrium analysis appears to be more appropriate for the prediction of crack depth."*

Cracking in Drying Soils (Morris (1992))

This paper appears to be the only one that suggests a relationship between the estimated depth of cracking of a clay soil and actual measured depth of desiccation cracks. The paper considers three approaches to calculating the depth of desiccation cracks. The three solutions were developed considering: 1) linear elasticity with decreasing horizontal stresses and tensile failure with the effective cohesion, c'= 0; 2) linear elastic fracture mechanics (LEFM) of the soil; and 3) using linear elasticity at the transition between tensile and shear failure of the soil where c'=0. As a soil surface dries, a matric suction, termed matrix suction in some literature, is developed at the interface of the soil, water and air. Consolidation of the soil occurs due to this suction stress. Shear strength is reduced as desiccation cracks form and this can cause slopes to be unstable. Micocracks form initially and grow into macrocracks under tensile stresses at the crack tip. Since suction is inversely proportional to the radius of voids, suction at the tip increases as particle size decreases. Fine grained soils are thus more impacted by desiccation. The paper states:

> *"The onset of cracking depends on the mineralogy of the soil, climatic conditions such as temperature and rainfall, and surface vegetation cover. At a selected strength level, plastic clays contain more water than lean clays. They therefore experience larger volumetric contractions on drying. They also have a larger effective cohesion **c'** and larger tensile strength t. This leads to wider, deeper cracks in plastic clays than in lean clays."*

Wide deep desiccation cracks occur in plastic soils where high temperatures exist during the dry season. Shrinkage cracks are usually vertical and extend almost to the depth of seasonal moisture change in a soil. The table below was abstracted from the paper and indicates the depth of observed desiccation cracks in Australia and Canada. Note that crack depth is reduced when groundwater contains salts as noted in Adelaide.

TABLE 1. Typical values associated with cracking natural soils in Australia and Canada

Morris, et.al. (1992)

Location	Suction ratio[a]	Depth of seasonal suction change (m)	Depth of cracks (m)
Australia			
Adelaide	1.2	4.0	1.8–2.0
Melbourne	1.2	2.0	2.0–3.0
Sydney	1.5	1.5	

NOTE: Table adapted from Australian Standard 2870.1-1988: Residential slabs and footings; [a] Ratio of suctions when most wet and most dry.

The seasonal change in water content, H_s, does not usually extend to the depth of the groundwater table, H_d. The matrix suction usually decreases from the ground surface where it is greatest, to the depth of water content change where it is 0, but variation is not linear. Unsaturated soil behavior can be explained by total stress, σ, the matrix suction, $u_a - u_w$, and void ratio, e, where u_a = pore air pressure and u_w = pore water pressure. Questions have arisen about the validity of effective stress concepts in partially saturated soils and Fredlund and Morgenstern (1976) showed that the shear strength can be expressed as:

$$\tau = c' + (\sigma - u_a) \tan \varphi' + (u_a - u_w) \tan \varphi^b$$

where the effective cohesion c' in this equation is independent of pressure and is often small, φ' describes how the strength increases with effective pressure, and φ^b describes how the strength increases with matrix suction. The apparent cohesion in an unsaturated soil, where the failure envelop intercepts the τ-envelop, may be written as:

$$c_{app} = c' + (u_a - u_w) \tan \varphi^b$$

And, the tensile strength of an unsaturated soil may be written as:

$$t = 0.5[c' + (u_a - u_w) \tan \varphi^b] \cot \varphi'$$

where the 0.5 is a constant which indicates that t is less than $c_{app} \cot \varphi'$. This relationship was used to develop the solutions in this paper.

Matrix suction in uncracked soils produces compressive stresses between soil particles causing consolidation. Desiccation cracks propagate vertically downward in the soil as it dries and matrix suction increases when tensile loading at the crack tips exceeds tensile strength.

The results of the three solutions are shown in the paper in a series of figures. The depth of cracking for groundwater at 10m, clay with $\varphi'=30^o$, matrix suction at the ground surface is shown for two values, S_o=50kPa=1044psf and S_o=100kPa=2088psf decreasing linearly with depth to the groundwater table, $\varphi^b = \varphi' - 5^o$, $\nu = 0.3$, c'=0 kPa, density of the soil, $\gamma=1835 kg/m^3$, and E=5MPa, are shown in Table A for the three solutions.

Table A – Results of Morris, et al (1992) and Wijesooriya, et al (2011) Research

	Calculated Depths of Desiccation Cracks, m	
Solution	S_o=50kPa	S_o=100kPa
Linear elasticity	1.8	3.0
Linear elastic fracture mechanics	4.2	6.5
Shear strength	2.0	3.3
UDEC shear failure	2.9	---

The report stated that the values for So=100kPa were in the range of the reported depths of desiccation cracks shown in Table 1 for the linear elastic and shear strength solutions. The linear elastic fracture mechanics results are unrealistically high. The relationship between depth of cracking and measured depths is not strong.

Prediction of Desiccation Crack Depths Allowing for Shear Failure (Wijesooriya, et al (2011))

This paper used the analyses developed in the previous paper by Morris, et al (1992) and added a solution using the computer code UDEC to predict desiccation cracking at shear failure using numerical analysis of soil suction development. The results are presented using the same theoretical assumptions and soil parameters, presented by Morris, et al (1992). The results are included in Table A above. It is noted that the results are midway between linear elastic fracture mechanics solution and the shear strength solution. The paper states:

> *"While analyzing cracks allowing shear failure, plastic behavior of the soil is taken into account which should reduce the crack depth due to plastic flow towards the crack."*

This paper didn't compare the numerical results to any actual measurements of desiccation crack other than to reference the Morris, et al (1992) paper.

Desiccation and Cracking Behavior of Three Compacted Landfill Liner Soils (Yesiller, et al (2000))

The study considered desiccation cracking of three soils used to construct compacted landfill liners in southeast Michigan. The significant properties extracted from the paper are included in Table B below.

Table B Soil Properties

Soil No.	Unified Soil Classification	Atterberg Limits LL	PI	Particle Distribution, % Sand	Silt	Clay	Hydraulic Conductivity, cm/s
1	CL-ML	22	6	25	45	30	1.1×10^{-8}
2	CL	29	16	19	39	42	7.8×10^{-8}
3	SC	17	6	68	21	11	1.0×10^{-7}

The authors found that fines content impacted the cracking behavior. Soils Nos. 1 and 2 cracked the most and Soil No. 3 cracked the least. Fines content was a better indicator of crack potential than Plasticity Index. Higher suction values were recorded in the soils with the highest fines content. The extent of cracking did not change significantly after the first wetting cycle which suggested that the fabric of the soil was altered after the first wetting and drying cycle. The changes in hydraulic conductivity were not measured during the study.

Flood Simulation Study of Retamal Levee, Lower Rio Grande Valley, Texas (Dunbar, et al (2007))

The Retamal levee is located on the north side of the Rio Grande River near McAllen, TX. The levee was constructed of highly plastic clays (CH) and is in an area that experiences droughts. The levee had experienced severe drought conditions for several years prior to a test pond being constructed to flood the severely desiccated landside levee surface 2004. A test trench was excavated in the levee to a depth of 10ft to observe the soil conditions. The clays were noted to be blocky and dry indicating a highly desiccated condition to a depth of 9ft, below which the clay was described as uniform and moist. There was a sharp contrast between the dry desiccated soil and the moist soil below.

Review of Engineered Covers for Waste Containment Research Report

Engineered Covers for Waste Containment: Changes in Engineering Properties and Implications for Long-Term Performance Assessment (Benson, et al (2011))

The Engineered Covers for Waste Containment study (Report) is 10 year effort and as such requires significant space for review. Report Section 2 Background: ACAP Test Sections describes the changes in engineering properties which occurred to covers for various disposal facilities in an Alternative Cover Assessment Program (ACAP). A total of 12 sites were evaluated and at 11 of these sites the soils were exhumed and tested after 4 to 9 years of service. The landfill covers considered in this study are not the same as levees from a technical perspective. Levees have an endless source of capillary moisture but these covers were placed over a capillary break. However, the permeability data for the various soil types in the study are beneficial.

Of particular interest are one site in an arid climate, four sites in semi-arid climates similar to Sacramento, one site in a sub-humid climate, and two humid sites. The sites of interest were located in Apple Valley, CA, Boardman, OR, Polson, MT, Sacramento, CA, Underwood, ND, Helena, MT, Cedar Rapids, IA and Albany, GA. These sites were selected for several reasons. Firstly, they were selected because the soils used to construct the barriers or store and release cross sections were similar to the materials available at the Southport Project site. The Cedar Rapids, IA and Albany, GA sites were added to better understand how these types of soils respond in humid climates. The sites that were not selected contained geomembrane barriers or had soils dissimilar to the Southport Project and so they were not included. Table 2.1 presents that climate and geometric data for the sites of interest. The following tables were abstractions from larger tables in the Report where the data for all 12 sites are presented.

Table 2.1. Climatic Characteristics and Slopes of ACAP Sites

Site Location	Service Life (yr)	Slope (%)	Average Annual Precipitation (mm/yr)	Avg. Precipitation / Potential Evapotranspiration	Climate
Apple Valley, CA	4.9	5	119	0.06	Arid
Boardman, OR	6.8	25	225	0.23	Semi-arid
Helena, MT	8.9	5	289	0.44	Semi-arid
Polson, MT	8.8	5	380	0.58	Sub-humid[1]
Sacramento, CA	6.0	5	434	0.33	Semi-arid
Underwood, ND	4.1	25	442	0.47	Semi-arid
Cedar Rapids, IA	5.7	25	915	1.03	Humid
Albany, GA	4.0	5	1263	1.10	Humid

[1] Based on Average Precipitation/Potential Evapotranspiration ratio

The reported stated in Section 6 Earthen Barriers and Storage Layers, page 6-1:

> *"Over the service life of a final cover, the hydraulic properties of earthen layers evolve due to the formation of soil structure in response to natural processes such as insect and animal burrowing, plant root growth, freeze-thaw cycling, wet-dry cycling, and distortion. These processes create cracks, fractures, and other larger-scale features that are generally referred to as macropores. Formation of macropores alters the network of pores controlling retention and*

movement of water in the field, which is reflected in changes in the hydraulic properties (e.g., permeability and soil water characteristic curves).”

These hydraulic properties increased without regard to climate, cover design, or service life.

Tables 6.1 below and Table 62 in the Appendix present the site characterization and as-built compaction data for the sites of interest. Four sites included conventional clay barriers and six included store-and-release cross sections. At the Underwood, ND site, both a conventional clay barrier and a store and release cross section were constructed. At the Sacramento, CA site two different thickness of sore and release cross sections were considered. The thickness of the covers varied from 1.75 to 8.0ft. Each barrier or store and release cross section was covered with 0.5ft of loosely placed topsoil except at the Cedar Rapids and Apple Valley, CA sites where a 2ft thick clay (CH) cover and a 1.0ft thick silty sand (SM) were installed, respectively, over the barriers (Albright, et al (2006a)) and at Boardman, OR where no topsoil was placed because of the silty material used to construct the store and release cross sections. This surface layer at Cedar Rapids, IA could be considered part of the barrier except that it was compacted to a very low density. The barriers or store and release cross sections classified GC at one site, SC at two sites, CL at five sites and CL-ML at two sites. The fines content ranged from a low of 31% in Albany, GA to a high of 93.2% in Polson, MT. The clay fraction varied from 12 to 30%. The PI and LL values varied from a low of 4 and 24 in Boardman, OR to a high of 47 and 67 in Helena, MT, respectively. The soils at Albany, GA and Helena, MT both classified SC and the thin section at Sacramento, CA classified GC. The soils at Boardman, OR and Polson, MT classified CL-ML and at the other four sites the materials all classified CL with a narrow range of PI and LL values, clay contents and fines contents. All of the sites of interest were vegetated with annual and perennial grass mixtures and had slopes of 5 or 25%.

Table 6.1. Site Characteristics and Soil Index Properties

	Thickness (ft)						Clay	Atterberg Limits	
Site Location	Surface	Barrier	Climate	Unified Soil Classification	Fines (%)	Silt %	Fraction < 2 m(%)	PI	LL
Store and Release Cross Sections									
Boardman, OR	0.0	6.0	Semi-arid	CL-ML	83.9	71.4	12.4	4	24
Helena, MT	0.5	4.0	Semi-arid	SC	44.5	14.7	29.8	47	67
Polson, MT	0.5	1.25	Sub-humid	CL-ML	93.2	75.1	17.9	7	28
Underwood, ND	0.5	2.5	Semi-arid	CL	66.6	41.4	25.2	23	39
Sacramento, CA	0.5	3.0	Semi-arid	GC	41.0	26.3	14.7	18	32
Sacramento, CA	0.5	8.0	Semi-arid	CL	71.8	53.9	17.9	22	39
Conventional Barriers									
Albany, GA	0.5	1.5	Humid	SC	30.8	7.8	23.0	13	28
Apple Valley, CA	1.0	1.0	Arid	CL	81.0	49.7	26.3	10	29
Cedar Rapids, IA		2.0	Humid	CL	52.2	29.9	22.3	19	33
Surface[1]	2.0			CH	76.0		28.0	18	53
Underwood, ND	0.5	4.5	Semi-arid	CL	60.8	36.6	25.2	21	38

The Report stated, *“Earthen barrier layers in conventional covers are placed with high effort to achieve low hydraulic conductivity, whereas storage layers are placed with modest effort to promote root development and to provide storage capacity.”* This distinction does not appear to hold for the selected sites. The barriers at Apple Valley, CA, Cedar Rapids, IA and Albany, GA were all constructed with conventional barriers and were compacted to low relative compaction values of 91.2, 92.7 and 93.0%, respectively, and the Underwood, ND store and release cross section had a high relative compaction of 97.2%.

The as-built saturated hydraulic conductivity, $k_{SA,}$ data are included in Table 6.3 in the Appendix. The Report uses the geometric mean rather than an arithmetic average value for hydraulic conductivity data. The geometric mean is used most often when the numbers are in different ranges or if there are a few very large numbers which affect the arithmetic average and make it meaningless. The most interesting hydraulic conductivity value is for Albany, GA, which had a very low value for an SC material.

After four to nine years of service life these sites were again tested to evaluate the effects of climate. Except for the Boardman, OR site, both field hydraulic conductivity tests using sealed double ring infiltrometers (SDRI) (effective diameter 1.69m) and large diameter boreholes (BH) (diameter 300mm) were performed and the resulting values are shown in Table 6.4 in the Appendix. At the Boardman, OR site only large diameter borehole tests were performed. Lysimeter scale tests (10m by 20m) were not performed on all of the sites of interest so these data are not included. The geometric mean value of these two types of field hydraulic conductivity tests, k_F, is also shown in Table 6.4 for each site. In the case of the Boardman, OR site the k_F value may have been obtained from large diameter borehole tests at two different test sections with similar properties but this could not be determined with certainty.

t-tests indicated that hydraulic conductivities from both field methods are statistically similar for store-and-release covers and conventional covers with clay barriers.

Laboratory tests were also performed on 305mm diameter samples trimmed from block samples and the data for the sites of interest are included in Table 6.5 in the Appendix.

These "large scale" tests were compared to the field tests and in seven of the eight sites of interest the permeability values of the large scale lab tests underestimated the field permeability values by the following factors: Boardman, OR, 4.2; Polson, MT, 2.1; Underwood, ND, 3.4; Sacramento, CA 5.8, Sacramento, CA, 8.1; Albany, GA, 3.8; Apple Valley, CA, 10.0; Cedar Rapids, IA, 35.0; and Underwood, ND, 4.9. The data for the Helena, MT site was reversed at 0.1. If only the SDRI hydraulic conductivity is compared the ratio is 0.7. The Report noted that, *"The hydraulic conductivity varies with scale of the volume of solid tested because the soil structure that is captured varies with the scale of the test."*

The geometric mean of all in-service hydraulic conductivity, k_{SI}, tests (SDRI, BH, and laboratory) was divided by the geometric mean of all as-built hydraulic conductivity tests, k_{SA}, and this ratio was defined as the hydraulic conductivity ratio. These results are shown in Table C along with other data gleaned from the previous tables so that an evaluation could be made of the general conclusions provided in the Report for the selected sites. This is an expansion of Table 6.6 included in the Report.

The general conclusions from the report are discussed below. Report <u>Section 6.5 Factors Affecting Changes in Hydraulic Conductivity stated the following.</u>

- *"---"healing" of structure in final covers is unlikely."*

The Report stated that there was no indication that healing of store and release cross sections and barriers occurred upon wetting. The Report confirmed earlier studies (Yong and Warkentin (1975), Yesiller, et al (2000), Albrecht and Benson 2001)) that indicted only one wet-dry cycle was needed to cause long term change in hydraulic conductivity. Each site experienced multiple wet-dry cycles during the 4 to 9 year study period.

Report <u>Section 6.7 Summary of Findings for Earthen Storage and Barrier Layers stated the following</u>.

- *"Similar saturated hydraulic conductivities were obtained with the BH and SDRI tests (within 10x). Saturated hydraulic conductivities based on peak flows in the lysimeters that were computed with and without accounting for degree of saturation generally bracketed the saturated hydraulic conductivities measured with the BHs and SDRIs."*
- *"Storage layers in the store-and-release covers had the highest saturated hydraulic conductivity followed by the clay barriers in conventional covers without a geomembrane. --- These differences reflect the higher compactive energy used to construct resistive barrier layers---. However, the hydraulic conductivities did not differ appreciably between cover types (< 10x), and the clay barriers in all of the conventional covers had higher saturated hydraulic conductivity than existed in the as-built condition."*
- *"Changes in the saturated hydraulic conductivity were similar regardless of climate (wet or dry) and no barrier type was found to be immune to an alteration in hydraulic conductivity. Wet-dry cycling appears to have a major role in the alterations in hydraulic conductivity."*
- *"Storage and barrier layers should be compacted to lower dry unit weight and at drier water contents to the extent practical to reduce the change in hydraulic conductivity that occurs while in service."*

Additional comments were included in Report <u>Section 10.1 Design Conditions</u>.

- *"For covers of typical thickness (< 3 m), the saturated hydraulic conductivity of earthen barrier and storage layers will increase over time in response to processes such as wet-dry and freeze-thaw cycling, with larger increases occurring in layers having lower as-built saturated hydraulic conductivity.---The changes occur regardless of climate, cover profile, or placement condition."*
- *"Smaller changes in saturated hydraulic conductivity occur in storage and barrier layers constructed with soils having lower clay content and fines containing a greater proportion of silt."*
- *"When practical, earthen storage and barrier layers should be constructed using fine-textured soils containing a broad range of particles (coarse and fine) with a modest amount of clay-size particles."*
- *"Soils classifying as SC, SM, ML, and SC-CL in the USCS are likely to be more resistant to changes in hydraulic properties over time compared to soils classifying as CL, CH, CL-CH, or CL-ML."*
- *"Earthen storage and barrier layers that are densely compacted tend to loosen over time and become more permeable. The porosity of most earthen storage and barrier layers evaluated in this study was between 0.35-0.45 when exhumed. Thus, to the extent practical, earthen storage and barrier layers should be compacted to a condition resulting in a porosity of approximately 0.40, which corresponds to a dry unit weight of approximately 15.5 kN/m3* (98.7pcf) *for a soil with a specific gravity of solids = 2.65.*

- *"Compaction wet of optimum water content should be avoided; compaction near optimum water content is recommended."*

The conclusion that store and release cross sections and barriers are degraded in semi-arid and arid climates as well as humid climates is counter intuitive. This is interesting since we as a BOSC have assumed that levees built of CH clays in Mississippi and Louisiana may be impacted by desiccation in summer heat but heal on wetting. The data in this Report suggest that this conclusion is not correct. The Report also states,

"This finding contrasts anecdotal reports, which suggest that clay barriers in drier climates are more readily or severely damaged by environmental exposure. The findings of this study indicate that significant alterations in hydraulic conductivity can occur in all climates."

The Appendix to the report also indicated that observations of the barrier at Underwood, ND contained roots and desiccation cracks from the ground surface to the bottom of the barrier to a depth of 5ft. Albright, et al (2006b) found the same result at Albany, GA where the depth to the bottom of the barrier was 2ft. Benson (2013) indicated in personal communication that the depth of desiccation cracking in the Sacramento area in CL clays was about 2 m (6.5ft). The degradation in hydraulic conductivity throughout the clay barriers and store and release cross sections suggests that for all sites studied the depth of desiccation cracking was the depth of covers, or up to 8ft.

The conclusions and characteristics noted above that were suggested to be predictors of either poor or good performance of store and release cross sections and conventional clay barriers, as indicated by the hydraulic conductivity ratio are discussed in the following paragraphs for the selected sites of interest. The data in Table C is used for this purpose.

Depth of Desiccation Cracking

The depth of desiccation can be concluded to be up to about 6 feet for the semi-arid sites considered that are vegetated. <u>For the Sacramento, CA area, the maximum depth of desiccation cracking was 6.5ft for CL clays.</u>

Changes in Hydraulic Conductivity

The hydraulic conductivity ratio_ratios of the semi-arid and arid sites were 7, 51, 395, 18, 156, and 395 and 1823. The ratios for the two humid sites were 28 and 2650. A lower ratio indicates less degradation of the soil hydraulic conductivity. There is also no indication that hydraulic conductivity is reduced by healing when wetting occurs in the wet-dry cycle. The hydraulic conductivity was increased throughout the depth of the convention clay barriers (1. to 4.5ft thick) and the store and release cross sections (1.25 to 8.0 ft thick) indicating that desiccation cracks extended to the bottom of these covers. <u>The hydraulic conductivity ratio values varied from 7 to 2650 and indicated that the hydraulic conductivity of soils at all sites were degraded from the as-built condition by about one to greater than three orders of magnitude.</u>

In the Sacramento, CA area the degraded hydraulic conductivity was found to be about $5x10^{-5}$ cm/sec after 6 years of service down from about $3x10^{-7}$cm/sec.

Climate

Excluding the Apple Grove, CA and Albany, GA sites the hydraulic conductivity ratios from 7 to 395. The geometric mean of the semi-arid and arid sites was 66 and for the humid and sub-humid sites were 33. The very high hydraulic conductivity ratio of the Apple Grove, CA and Albany, GA sites may have occurred because they were the thinnest barriers at 1.0 and 1.5ft. All other barriers and store and release cross sections exceeded a thickness of 2.5ft if the CH clay cover is included as part of the CL barrier at the Cedar Rapids, IA site. The geometric mean of all of the semi-arid and arid sites in the research report was 93 and for the humid and sub-humid sites it was 89. Based on this assessment the soils at the semi-arid and arid sites were slightly more degraded by the wet-dry cycles they experience than the soils in humid and sub-humid climates, but all soils in both humid and semi-arid climates experience degradation.

Lower Clay Content

The only barrier or store and release cross section with both a low hydraulic conductivity ratio and low clay content was the Boardman, OR site with a ratio of 7 and a clay content of 12.4% compared to the average of all sites of 21.5%. The range of clay contents was from 12.4 to 29.8%. The site with the highest clay content, Helena, MT, also had a reasonably low ratio of 51. Thus, as applied to the selected sites, low clay content is not considered a very good indicator of a low hydraulic conductivity ratio.

Higher Silt Content

Two sites had silt contents higher than 70%: Boardman, OR with 71.4% and Polson, MT with 75.1%. These sites had the lowest hydraulic conductivity ratio of 7 at Boardman, OR and a relatively low ratio of 39 at Polson, MT. The lowest silt content occurred at the Albany, GA site which also had the highest hydraulic conductivity ratio. Based on these results it can be concluded that a silt content greater than 70% likely will produce a low hydraulic conductivity ratio and a less degraded soil.

Soils Classifying as SC, SM, ML, and SC-CL are Likely to be More Resistant to Changes in Hydraulic Properties Over Time

This criterion was upheld for two semi-arid sites considered: they classified SC and GC where the hydraulic conductivity ratio values were 51 and 18, respectively. However two of the lowest ratios were reported at Boardman, OR and Polson, MT which was constructed of CL-ML material, one of the material types suggested to be excluded. The other semi-arid sites were constructed of CL materials and had ratios of 395, 156, and 214, respectively. The fact that the Albany, GA site was constructed of SC soils didn't support this criterion. This criterion appears to provide some assurance that degradation of the soil will be limited if CL-ML soils are included even considering the results of the Albany, GA site.

Lower Compacted Dry Density

The Report recommends an as-built dry unit weight of about 99pcf to reduce degradation of liners. The Albany, GA site dry unit weight was 106.9pcf and experienced the greatest reduction in hydraulic conductivity with a ratio of 2650, but the Cedar Rapids, IA site had a dry unit weight of 113.4pcf and a reasonable ratio of 28. If the range of dry unit weights comparable to the porosity range of 0.35 to 0.45, or 91 to 107pcf is considered, then three sites performed well with ratios of 18, 39 and 395. The site with the lowest hydraulic conductivity ratio of 7, Boardman, OR, had a dry unit weight at the lower limit of this range. This criterion does not appear to hold much promise to project satisfactory long term performance. Relative compaction may be more effective at predicting lower hydraulic conductivity ratios. The sites with the lowest ratios of 7, 18, 28, 39 and 51 had relative compaction values below 94% at 83.2%, 93.6%, 92.7%, 92.0% and 91.1%. But, the Albany, GA and Apple Valley, CA sites had a relative compaction at 93.0% and 91.2% and yielded very high ratios of 2650 and 1823. These were also the thinnest barriers in the study. The other sites with relative compaction values above 94% all had higher ratios ranging from 156 to 395. Low relative compaction values, in the 90 to 94% range, appeared to generally yield lower hydraulic conductivity ratios.

Compaction at Optimum Water Content

The soils at three of the five sites with low hydraulic conductivity ratios were compacted at 2.8 to 4.2% above optimum moisture content. The Albany, GA site was compacted at optimum moisture content and produced the most degradation in hydraulic conductivity. This criterion does not appear to be effective at predicting low hydraulic conductivity ratios

Review of Stability References

Long Term Failure in Compacted Clay Slopes (Templeton, et al (1984))

This paper describes the process of evaluating the depth of surficial cracking in Mississippi River levees in Mississippi and Louisiana and the resulting slough slides which occurred many years after the levees were constructed. The authors indicated that the cracking extended to depths of 5 to 7ft and resulted in loss of strength over time in these highly plastic clays. Back analysis of these surficial failures indicated that the original peak strengths that existed at the time of construction had been reduced significantly at the time of failure.

Design Deficiency Corrections, Alton to Gale Levee Organized Levee Districts, Illinois and Missouri (O'Hara (2010)

This paper describes the process of repairing shallow levee slides along a 90 mile section of Mississippi River levees in Illinois and Missouri originally constructed of highly plastic clays (CH). The slides have been occurring since 1958 and have been repaired in the past by methods which were not successful. In the late 1990's the upper 7ft of the levee material was removed and replaced with 7ft of low to medium plastic materials. The depth of removal was about 2 feet below the base of previous slides. No new slides have occurred since this repair effort was completed.

Report of the Workshop on Shear Strength for Stability of Slopes in Highly Plastic Clays (Duncan, et al (2011)

This work shop consisted of 57 invited attendees including twelve members of the Corps of Engineers, Drs. Mike Duncan and Jim Mitchell, Virginia Tech, Dr. Stephen Wright, University of Texas, Dr. Les Harder, HDR, Dr. Ross Boulanger, UC Davis, BOSC members Mr. George Sills and Dr. Ray Martin, and many more highly qualified people. The purpose was in part to *"... discuss the current state of practice for design of slopes in highly plastic clays in various parts of the United States"*

Duncan reviewed the literature which has led to our current understanding of the use of fully softened drained shear strength (FSS) in analyses of the stability of embankments constructed of highly plastic clays. He noted that Kayyal and Wright (1991) found that the strength of highly plastic clays in Texas embankments was reduced over time to the FSS as the result of rainwater entering desiccation cracks developed during periods of droughts which caused the clays to soften. They along with Day and Axten (1990) in southern California, McCook (1997) in Oklahoma found that the depths of the slides along embankments constructed of highly plastic clays were very shallow and ranged from 3 to 6 ft measured normal to the slopes.

Wright reviewed findings from projects in Texas from the Beaumont (LL=73, PI=52, clay fraction = 47%) and Paris (LL=80, PI=58, clay fraction = 58%) clays which indicated the FSS envelop was curved as shown in the figure below.

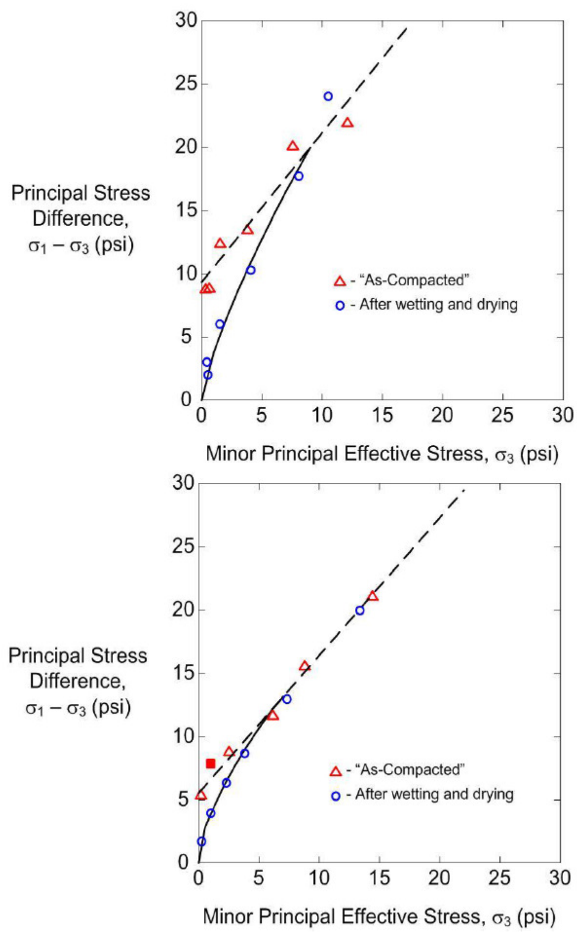

Figure 5 Strengths of Beaumont clay (top) and Paris clay (bottom) before and after 10 cycles of wetting and drying.

The workshop was divided into four groups and the Softening Process Group, of which Martin was participant, concluded the primary mechanism of softening for compacted embankment fills was caused by desiccation which reduced,

> "*---lateral pressures within the desiccated zone, and allowing infiltration of rain*---such that---*the desiccated zone (compared with intact zone)---absorbs additional water and becomes softer and weaker than the intact zone.*"

This Group also agreed that, *"---softening can occur in clays of lower plasticity, but is most pronounced in highly plastic clays"* and increases with higher Plasticity Index, clay size fraction and activity and lower silt and sand content.

The Use of Fully Softened Strength in Stability Analyses Group, of which Sills was a participant, concluded that *"FSS failures in embankments tend to be shallow*---and that there was a---*marked change in consistency---below the softened zones on exhumed slides ---."*

Finally, there was general consensus by the workshop participants as a whole that,

> *"Fully softened shear strengths should be used, with appropriate long-term pore pressures, for analysis of the stability of shallow potential slides in embankments where wet/dry climate cycles are likely to produce significant desiccation cracking.*

Summary of Literature Reviewed

General References

The following comments are presented as a summary of the findings of the general references.

- Mitchell (1993) indicated that the presence of highly active clay particles in larger quantities promotes crack formation to a greater extent than soils with less active clay mineral and a lower percentage of clay size particles. Fine-grained soils have smaller pores and thus are more susceptible to the development of cracks than coarse-grained soils because they develop higher suction stresses.

- Kodikara, et al (2000) reviewed the results of Corte and Higashi (1960) and found that the desiccation cracking water content decreased as the desiccation rate increased. The desiccation rate was varied to by control of temperature and humidity.

- Tang (2007, 2009?) found that the relationship between the surface crack ratio, *R*sc and water content, which was defined as the cracking curve, stabilized as the water content reached the shrinkage limit, SL, of the soil. The cracking curve was found to reflect the shrinkage properties of the soil. The study also found that as the temperature rises, the cracking water content, *w*c, rises and the residual water content at equilibrium declines.

- Inci (2008) reported the factors that affect desiccation cracking of soils: clay mineralogy, clay content, compaction conditions, drying process, wetting and drying cycles, soil particle orientation, unit weight, and pore fluid. He also noted that Daniel (1991) found that shrinkage is the highest when the plasticity index is high and shrinkage limit is low and that Daniel and Wu (1993) recommend that clayey sand be used for liner construction at arid sites because it combines lower hydraulic conductivity and lower shrinkage potential. They also recommend compaction at lower water contents with high relative compaction to reduce cracking in arid areas. Inci also reported that Warkentin (1975) found that irreversible fabric changes occurred during the first drying cycle in a clay soil. The author also concluded that the Finite Element Method was an effective tool to model desiccation cracking.

- Taha, et al (2011) reported that Omidi (1993), Albrecht and Benson (2001), Osinubi and Eberemu (2010), Osinubi and Nwaiwu (2008), Harianto, Hayashi *et al*., (2008), and Puljan (2010) found that shrinkage strain depended on three main parameters: molding water content, compaction effort (dry density), and soil plasticity index. They concluded that as molding water content decreases and molding density increases shrinkage of the soil decreases. They also noted that lime treatment of soils is not effective in reducing desiccation (Guney, et al., (2007)) but that cement treatment of clay soils with a PI <20 was effective (Walker (1995)) and that fiber treatment of silty soils was effective in reducing cracking (Rifai and Miller (2009)).

- Fleming, et al (1992) found that lime treatment of highly plastic CH clays was effective in ameliorated the properties of these clays when repairing shallow surficial slides up to 5 to 7ft deep along hundreds of miles of Mississippi River levees in Mississippi and Louisiana. The lime treated soils classified CL and ML after treatment. No slides were reported in slopes constructed of low plasticity clays classified CL or with PI<27. The final repair included a 3ft thick cover layer of lime treated clay which proved effective in produced a long term repair. None of the repaired slides have failed in the intervening 20+ years.

- Lau (1987) described two approaches to evaluating the volume change behavior of unsaturated clay soils: the first was derived using volume change behavior (elastic equilibrium analysis) and the second using shear strength behavior (plastic equilibrium analysis). He found that desiccation cracking is the result of soil volume reduction and therefore an "*elastic equilibrium analysis appears to be more appropriate for the prediction of crack depth.*"

- Morris, et al (1992) found that solutions using linear elastic behavior and an approach which relates cracking to a transition between tensile and shear failure produced cracks of comparable depths. The paper compares the depths from the analysis to typical crack depths and there is a reasonable comparison, but, the analyses were not completed for specific field conditions.

- Wijesooriya and Kodikara (2011) expanded the analysis completed by Morris, et al (1992) using a numerical solution and the same properties used by Morris and found that crack depths were somewhat greater but again there is no comparison to actual field crack depths.

- Yesiller, et al (2000)) found that fines content was a better indicator of crack potential than Plasticity Index and that the highest suction values were recorded in the soils with the highest fines content. The extent of cracking did not change significantly after the first wetting cycle which suggested that the fabric of the soil was altered after the first wetting and drying cycle. The changes in hydraulic conductivity were not measured during the study.

- Dunbar, et al (2007) present data on the depth of desiccation on a levee located on the Rio Grande River near McAllen, TX. The levee was constructed of highly plastic clays (CH) and is in an area that experiences severe droughts. A test trench was excavated to observe the soil conditions. The clays were noted to be blocky and dry indicating a highly desiccated condition to a depth of 9ft, below which the clay was described as uniform and moist.

Engineered Covers for Waste Containment Research Report

This research report was reviewed but only the sites with conventional clay barriers without geomembrane covers and store and release cross sections were considered. The following comments pertain to assessment of these specific sites by the BOSC and not the authors of the report.

- The landfill covers considered in this study are not the same as levees from a technical perspective in that they do not have an endless source of capillary moisture from the interior of the levee.
- The depth of desiccation cracking at the semi-arid sites considered is estimated to be about up to about 6.5ft, the maximum amount reported in the Sacramento, CA area.
- Desiccation is caused not only summer heat but by plants that have root systems that penetrate up to 6 to 8ft in search of water,
- The hydraulic conductivity of soils at all sites was degraded from the as-built condition by about one to greater than three orders of magnitude.
- The semi-arid sites were slightly more degraded by wet-dry cycles than the soils in humid climates, but all soils in both humid and semi-arid climates experience degradation as measured by the hydraulic conductivity ratios.
- The degraded hydraulic conductivity for all sites considered ranged from 10^{-4} to 10^{-6} cm/sec and was $5x10^{-5}$ cm/sec for the CL clay in Sacramento, CA (as-built $k=3x10^{-7}$ cm/sec).
- Silt content > 70% and soils classified SC, GC, and CL-ML may be effective at reducing the degradation of hydraulic conductivity.
- Low relative compaction values, in the 90 to 94% range, appeared to generally yield lower hydraulic conductivity ratios.

Stability References

The following comments are presented a summary of findings from stability references.

- Templeton, et al (1984) describe the process of evaluating the depth of surficial cracking in Mississippi River levees in Mississippi and Louisiana and the resulting slough slides which occurred many years after the levees were constructed. The authors indicated that the cracking extended to depths of 5 to 7ft and back analyses indicated significant strength loss had occurred over time in these highly plastic CH clays.

- O'Hara (2010) describes the repair of 90 miles of Mississippi River levees in Illinois and Missouri in the 1990's that were constructed of CH clays by removing the upper 7ft of the levee material and replacing it with 7ft CL clays. The depth of removal was about 2 feet below the base of previous slides. No new slides have occurred.

- Duncan, et al (2011) reported on a workshop that was convened in part to *"discuss the current state of practice for design of slopes in highly plastic clays in various parts of the United States."* He noted that Kayyal and Wright (1991) found that the strength of highly plastic clays in Texas embankments was reduced over time to the full softened drained shear strength (FSS) as the result of rainwater entering desiccation cracks developed during periods of droughts which caused the clays to soften. They along with Day and Axten (1990) in southern California, and McCook (1997) in Oklahoma found that the depths of the slides along embankments constructed of highly plastic clays were very shallow and ranged from 3 to 6ft measured normal to the slopes. Wright, et al (2007) indicated that the FFS failure envelops were non-linear and passed through the origin with c'=0psf. The workshop was divided into four groups and the Softening Process Group concluded the primary mechanism of softening for compacted embankment fills was caused by desiccation which reduced lateral pressures and allowed softening due to adsorbed rainwater. This Group also agreed that softening was most pronounced in highly plastic clays. The Use of Fully Softened Strength in Stability Analyses Group concluded that FFS failures tend to be shallow and that the soils below the weathered zone change consistency. The group knew of no published case studies of deep seated slides due solely to softening of embankment fill. Finally, there was general consensus by the workshop participants that, FFS strengths were appropriate for analysis of shallow embankment slides where wet-dry climate cycles caused significant desiccation.

Conclusions and Recommendations

This literature review was performed to evaluate whether a thin soil cover layer could be used over highly plastic CH clay soils to protect the CH soils from: 1) desiccation cracking and resulting weathering, and 2) reduction to the fully softened strength with the subsequent potential for shallow maintenance type slope instability. This approach would allow use of greater quantities of available CH clay for embankments construction.

Conclusions

No specific literature could be located related to the problem as defined above, however there is a significant body of literature that is helpful in developing recommendations for implementing the proposed concept. The conclusions drawn from the literature review follows.

- Desiccation cracking of fine grained soils occurs because of wet-dry climate cycles and is most evident in soils with highly active clay particles in larger quantities, e.g.CH clays.
- The depth of cracking is increased with higher temperatures and higher plasticity.
- As molding water content decreases and molding density increases shrinkage of the soil decreases.
- Irreversible fabric changes occurred during the first drying cycle in clay soils and healing of these fabric changes is unlikely based on increased permeability in the long term.
- The degradation of the hydraulic conductivity of clay soils due to wet-dry climate cycles occurs without regard to climate – humid or arid.
- The increase in hydraulic conductivity has been found to range from about one to greater than three orders of magnitude again without regard to climate.
- The degraded hydraulic conductivity of a Sacramento store and release cross section cover after 6 years of service was about $5x10^{-5}$ cm/sec for a CL clay (LL=39, PI=22); the as-built hydraulic conductivity value was $k=3x10^{-7}$ cm/sec.
- Some efforts have been made to estimate the depth of desiccation cracking analytically and numerically but research with direct correlation to actual measured crack depths was not found.
- The references cited indicate that the depth of potential softening of CH clays and instability due to fully softened conditions for embankment slopes in Texas, Oklahoma, Mississippi, Louisiana and California is limited to 3 to 7 ft, measured normal to the slope.

- The depths of 3 to 7ft are believed to represent the range of desiccation crack depths based on observations of slide scarps in Mississippi and Louisiana which indicated that a distinct change in consistency occurred with no weathering of soils below slide depths.
- The recorded depth of desiccation along the Rio Grande River near McAllen, TX was 9ft.
- The recorded depth of desiccation cracking in Sacramento for a CL clay was about 6.5 ft.
- Lime treatment was found to be effective at reducing the LL and PI of CH clays which were used to construct a 3ft thick cover layer over CH clay backfill to repair shallow levee slides along the Mississippi River.
- Lime treatment ameliorates CH clays such that they classified CL or ML after treatment.
- Other CH clay levees along the Mississippi River have been repaired by replacing the upper 7ft of CH clays with CL clay after the CH clays had experienced shallow slides.
- Soils with silt contents greater than about 70% may be effective at reducing the degradation of hydraulic conductivity due to collapse of silt particles upon wetting into desiccation cracking.
- Soils classified SC, GC, and CL-ML appear to less impacted by an increase in permeability due to desiccation cracking and this implies that weathering and associated softening is less significant in these soils.
- The previous conclusion was verified by the fact that no shallow slides have been reported related to desiccation cracking and softening of clays classified CL.
- Soils classified SC or CL-ML would be even less likely to develop fully softened conditions than CL clays because the properties critical to development of a fully softened condition (high clay content, highly active clay, more extensive desiccation) are less prominent in these soils.
- The fully softened drained shear strength is the appropriate value to use to assess the stability of shallow failure surfaces in embankments of highly plastic CH soils.

Recommendations

Based on the above conclusions the following recommendations are provided for consideration.

- The Corps of Engineers requirement that CH clays be encapsulated with a minimum of 10ft of CL clay with a maximum LL=45 is conservative and consideration should be given to replacing this requirement with a thinner encapsulating layer.
- A thin cover layer should be designed to replace the 10ft thick layer presently required by the Corps with the thickness depending upon the type of material to be used in the cover.
    - A 3.5ft thick cover layer is considered adequate if it is constructed of CL-ML material with maximum LL = 30 and PI=7
    - A 4.5ft thick cover layer is considered adequate if it is constructed of CL material with a maximum LL=40 and PI=15
- This cover layer and CH clay below to a depth of 6 to 7ft should be assumed to have a hydraulic conductivity as follows.
    - For a 3.5ft thick cover layer of CL-ML material: one order of magnitude higher then presently recommended
    - For a 4.5ft thick cover layer of CL material: 2.2 orders of magnitude higher then presently recommended
- The shear strength of the cover layer and underlying CH clay to a depth of 6 to 7ft should not be modified from present recommendations.
- If the decision is made to use CH clays throughout the embankment including the surface then a fully softened drained strength should be used in stability analyses to a depth of 6 to 7ft.

- The effective cohesion, c', should be set to 0psf.
- A curved failure envelope should be developed based on existing shear strength data and failure envelopes defected downward to the origin below 1000psf effective normal pressure.

Discussion

Based on the conclusions proved above, the Corps policy concerning encapsulation of CH clay with 10ft of CL clay is conservative and expensive.

The concept of placing a thin cover layer of better quality material over CH clays has been used successfully by the Corps in other districts to remediate problems with shallow slough slides in the CH clays. This approach is based on known geotechnical engineering principles. The better quality material will have the impact of reducing the depth of desiccation that would occur if the CH clays were at the surface because the pore spaces are larger in the cover materials than the underlying CH clays and because the CH clays will continue to have access to capillary moisture from within the levee core. The more extensive desiccation (width and depth of cracks) that would occur if the CH clays were at the surface, will be limited by the higher strength cover material, much like a crushed stone bridge layer reduces the impacts of an underlying soft layer due to its higher strength.

The proposed 3.5ft thick CL-ML cover layer will allow some desiccation of the underlying CH clays but will not be impacted by softening. On wetting some of the overlying low plasticity CL-ML soils will be eroded into and cog these cracks reducing the potential for softening of the CH clays. It is anticipated that desiccation cracks will be limited to a depth of about 2 or 3ft into the CH clays. If the softening does not progress then strength reduction will not occur, and the levees will not experience shallow surface instability if properly designed.

The thicker cover layer allows use of CL clays. The CL clays will not be impacted by softening and only about 1 to 2ft of the underlying CH clays will be impact by desiccation. This is not considered sufficient to create enough softening in the CH clays to negatively impact stability. The advantage of this approach is that the surface layer is much less erodible.

The increase in hydraulic conductivity to a depth of 6 to 7ft is based on the Benson, et al (2011) findings. The recommended CL-ML cover hydraulic conductive increase of one order of magnitude is based on the findings for Underwood, ND site. The recommended CL cover increase in hydraulic conductivity of 2.2 orders of magnitude was based the findings from Sacramento.

Finally, if the decision is made to us CH clays throughout the levee embankment then the recommendations related to fully softened drained shear strength from the Duncan, et al (2011) workshop report should be followed.

We believe that these recommendations will provide safe and resilient levees for the project.

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Appendix

Table 6.2. Average As-built Compaction and Water Content

	Standard Proctor Values		As-Built Values			
	Max. Dry Unit Weight	Optimum Water Content	Dry Unit Weight	Water Content	Relative Compaction	As-Built
Site Location	γ_{dmax} (kN/m^3)	w_{opt} (%)	γ_{dc} (kN/m^3)	w_c(%)	(%)	w_c-w_{opt} (%)
Store and Release Cross Sections						
Boardman, OR	17.1(108.9)	17.1	14.2(90.4)	20.5	83.0	3.4
Helena, MT	15.4(98.0)	22.4	14.0(89.1)	26.6	90.9	4.2
Polson, MT	16.4(104.4)	18.9	15.1(96.1)	16.9	92.0	-2.0
Underwood, ND	16.6(105.6)	19.4	16.1(102.5)	18.3	97.1	-1.1
Sacramento, CA	17.6(112.0)	16.5	16.5(105.0)	16.1	93.8	-0.4
Sacramento, CA	16.0(101.8)	21.7	15.3(97.4)	21.3	95.7	-0.5
Conventional Barriers						
Albany, GA	18.1(115.2)	15.7	16.8(106.9)	15.8	93.0	0.1
Apple Valley, CA	19.3(122.8)	12.5	17.6(112.0)	15.2	91.2	2.7
Cedar Rapids, IA	19.2(122.2)	12.2	17.8(113.3)	15.0	92.7	2.8
Surface(x)	13.1(83.4)	31.1	11.8(75.1)	34.6	90.0	3.5
Underwood, ND (5ft)	17.4(110.8)	16.8	17.6(112.0)	15.1	101.1	-1.7

Table 6.3. As-built Hydraulic Properties (from block samples - saturated falling head tests)

	Saturated Hydraulic Conductivity	
	(geometric mean)	Number of
Site Location	k_{SA} (m/s(cm/s))	Tests
Store and Release Cross Sections		
Boardman, OR	$1.3\text{x}10^{-7}(1.3\text{x}10^{-5})$	36
Helena, MT	$1.5\text{x}10^{-9}(1.5\text{x}10^{-7})$	16
Polson, MT	$4.0\text{x}10^{-9}(4.0\text{x}10^{-7})$	8
Underwood, ND	$1.8\text{x}10^{-9}(1.8\text{x}10^{-7})$	2
Sacramento, CA (3ft)	$1.9\text{x}10^{-8}(1.9\text{x}10^{-6})$	6
Sacramento, CA (8ft)	$3.2\text{x}10^{-9}(3.2\text{x}10^{-7})$	18
Conventional Barriers		
Albany, GA	$4.0\text{x}10^{-10}(4.0\text{x}10^{-8})$	5
Apple Valley, CA	$1.7\text{x}10^{-10}(1.7\text{x}10^{-8})$	8
Cedar Rapids, IA	$1.7\text{x}10^{-10}(1.7\text{x}10^{-8})$	8
Underwood, ND (5ft)	$1.2\text{x}10^{-9}(1.2\text{x}10^{-7})$	4

Table 6.4. Field Hydraulic Conductivity (tests performed at end of study, in-service)

Site Location	Sealed Double Ring Infiltrometer k_{SWRI} (m/s(cm/s))	Two Stage Borehole k_{BH}(m/s(cm/s))	Field Hydraulic Conductivity (geometric mean) k_F (m/s(cm/s))
Store and Release Cross Sections			
Boardman, OR	--	$1.3x10^{-5}(1.3x10^{-3})$	$2.0x10^{-6}(2.0x10^{-4})$
Helena, MT	$1.4x10^{-7}(1.4x10^{-5})$	$2.6x10^{-8}(2.6x10^{-6})$	$1.8x10^{-8}(1.8x10^{-6})$
Polson, MT	$8.9x10^{-8}(8.9x10^{-6})$	$1.8x10^{-7}(1.8x10^{-5})$	$7.7x10^{-8}(7.7x10^{-6})$
Underwood, ND	--	$1.2x10^{-6}(1.2x10^{-4})$	$1.2x10^{-6}(1.2x10^{-4})$
Sacramento, CA (3ft)	$2.1x10^{-6}(2.1x10^{-4})$	$5.8x10^{-7}(5.8x10^{-5})$	$7.5x10^{-7}(7.5x10^{-5})$
Sacramento, CA (8ft)	$7.8x10^{-7}(7.8x10^{-5})$	$1.4x10^{-6}(1.4x10^{-4})$	$1.3x10^{-6}(1.3x10^{-4})$
Conventional Barriers			
Albany, GA	$2.0x10^{-6}(2.0x10^{-4})$	$1.7x10^{-6}(1.7x10^{-6})$	$1.8x10^{-6}(1.8x10^{-4})$
Apple Valley, CA	$5.6x10^{-7}(5.6x10^{-5})$	$\cancel{2.1x10^{-5}(2.1x10^{-3})}$[1]	$5.6x10^{-7}(5.6x10^{-5})$
Cedar Rapids, IA	$1.3x10^{-8}(1.3x10^{-6})$	$3.5x10^{-8}(3.5x10^{-6})$	$2.8x10^{-8}(2.8x10^{-6})$
Underwood, ND (5ft)	--	$4.9x10^{-7}(4.9x10^{-5})$	$4.9x10^{-7}(4.9x10^{-5})$

[1] Not used because these values are more than two orders of magnitude higher than the SWRI and lysimeter tests.

Table 6.5. Saturated Hydraulic Conductivity Measured in Laboratory (tests performed at end of study, in-service)

Site Location	Hydraulic Conductivity K_S(m/s(cm/s))
Store and Release Cross Sections	
Boardman, OR	$4.8x10^{-7}(4.8x10^{-5})$
Helena, MT	$1.9x10^{-7}(1.9x10^{-5})$
Polson, MT	$1.6x10^{-7}(1.6x10^{-5})$
Underwood, ND	$3.5x10^{-7}(3.5x10^{-5})$
Sacramento, CA (3ft)	$1.3x10^{-7}(1.3x10^{-5})$
Sacramento, CA (8ft)	$1.6x10^{-7}(1.6x10^{-5})$
Conventional Barriers	
Albany, GA	$3.6x10^{-7}(3.6x10^{-5})$
Apple Valley, CA	$5.6x10^{-8}(5.6x10^{-6})$
Cedar Rapids, IA	$8.0x10^{-10}(8.0x10^{-8})$
Underwood, ND (5ft)	$1.0x10^{-7}(1.0x10^{-5})$