WEST Sacramento Levee Improvement Program
Board of Senior Consultants

Comments and Recommendations
Following Meeting No. 6
Of the Board of Senior Consultants
On February 21 - 22, 2012

Report Prepared by:

Board of Senior Consultants:

Dr. David T. Williams
Dr. Ray E. Martin
Mr. George L. Sills

April 10, 2012
April 10, 2012

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

Dear Mr. Ruzich:

I. Introduction

This report presents the comments and recommendations for the West Sacramento Levee Improvement Program (WSLIP) by the Program’s Board of Senior Consultants (BOSC) following a meeting held for, and with, the BOSC on February 21 - 22, 2012. This meeting was the sixth formal meeting of the BOSC and was held to provide detailed information by the City’s consultants on the WSLIP portion of the project known as Southport. As with other portions of the WSLIP, the analyses and designs being developed are part of the effort to provide 200-year flood protection to the Program.

During the meeting, presentations were made to the Board regarding the major subjects in the agenda (the agenda is Attachment 1).

The BOSC also responded to the Charge to the Board (Attachment 2) and addressed the instructions to the Board (Attachment 3).

The detailed meeting notes, taken by MBK Engineers and HDR, are in Attachment 4. This Attachment documented the BOSC interaction with the presenters and designers. Attachment 5 documents the BOSC comments on the various reports given to the BOSC prior to the February 21 – 22, 2012 meeting in additions to the comments that came out during the meeting. Note that some of the comments of the documents were addressed in the meeting presentations and may have been addressed to the BOSC’s satisfaction; however, these were left in to express the BOSC’s questions and concerns.

II. General Comments

The BOSC is pleased with the presentations on the preliminary status of the project, to understand the thought processes of the City’s consultants, and with the opportunity to work with and provide input to WSAFCA’s design team at such an early stage of the project.

IV. Closing Remarks

The Board appreciates the efforts of the design team members who prepared and presented numerous valuable summaries of the assumption for the proposed design. The various presentations and discussions were informative to the Board and helped further clarify the design teams’ thoughts and philosophy.

The Board looks forward to future meetings, briefings, and discussions on this project and is excited about the next phase of the project.
Very truly yours,

West Sacramento Levee Improvement Program
Board of Senior Consultants

Dr. David T. Williams, P.E. CFM.  Mr. George L. Sills, P.E.

Dr. Ray E. Martin, P.E.

Attachments:
Attachment 1: Meeting Agenda
Attachment 2: Charge to the Board
Attachment 3: Instructions to the Board
Attachment 4: Minutes of BOSC #6 meeting (by MBK Engineers)
Attachment 5: BOSC Comments on Documents provided for Review
Attachment 1: Meeting Agenda

WEST SACRAMENTO AREA FLOOD CONTROL AGENCY
MEETING AGENDA

WEST SACRAMENTO LEVEE IMPROVEMENT PROGRAM
BOARD OF SENIOR CONSULTANTS
MEETING NO. 6

Date: February, 21-22, 2012
Time: 8:00 am to 5:00 pm
Location: West Sacramento Community Center  1075 West Capitol Avenue
           Parking  1020 West Capital Ave.
           West Sacramento, CA 95691  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 DESIGN BACKGROUND  8:30 AM - 9:00 AM
   ▪ TO 1 (HDR)
   ▪ TO 2 (HDR)
   ▪ TO 3 (HDR)
   ▪ TO 4 (HDR)

BREAK  9:00 AM – 9:15 AM

III. OVERVIEW OF PREFERRED ALTERNATIVE  9:15 AM – 10:15 AM
   ▪ Deficiencies
   ▪ Design Objectives
   ▪ Corrective Measures by Segment

BREAK  10:15 AM – 10:30 AM

IV. DEVELOPING PROJECT BASIS OF DESIGN  10:30 AM – 11:30 AM
   ▪ Design Criteria
   ▪ Site Civil
   ▪ Geotechnical
   ▪ Geomorphic
   ▪ Groundwater
V. LUNCH & GEOMORPHIC (Catered in Community Center) 11:30 AM – 1:00 PM
   • Presentation (Bowles)
   • Discussion

BREAK 1:00 PM – 1:15 PM

VI. GEOTECHNICAL 1:15 PM – 5:00 PM
   • Presentation (Lokteff)
   • Discussion

DAY 2

VII. SITE CIVIL 8:00 AM – 9:30 PM
   • Presentation (Vecchio)
   • Discussion

BREAK 9:30 AM – 9:45 AM

VIII. GROUNDWATER 9:45 AM – 11:30 PM
   • Presentation (Browning)
   • Discussion

IX. LUNCH/BOSC Working Session 11:30 AM – 1:00 PM
   • Note: Design team to be available, as needed, to address BOSC questions

X. EIP SITE VISIT or CONT. BOSC WORKING SESSION 1:00 PM – 2:30 PM

XI. QUALITY MANAGEMENT PLAN 2:30 PM – 3:00 PM
   • Presentation (Blake)
   • Discussion

XII. REVIEW COMMENTS 3:00 PM – 4:30 PM
   • Overview of Comments
   • Comment Clarification & Discussion
   • Summary of Actions for Comment Resolution

XIII. CONCLUSIONS & ACTIONS 4:30 PM – 5:00 PM
   • Review 15% plan set & Technical Approach Memo
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 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.

All portions of the Charge to the Board have been taken into consideration by the Board of Senior Consultants and the associated activities have been included in the Board’s report as part of this package.
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 report for the meeting addresses issues raised based on review of 1) the design memoranda and 2) presentations at the BOSC meeting. If these comments are addressed, the various memoranda should provide adequate basis of design documentation.

2. What mitigation measure could be implemented that would allow for the waterside borrow operation to encroach on the USACE standard 100 foot waterside bench? What additional analysis should be performed?

   The seepage analyses that have been completed and that are planned should be adequate assuming 1) the final field investigation provides adequate coverage to reasonably identify the subsurface conditions, and 2) the laboratory testing of recovered soil samples is adequate to reasonably define the permeability of the various soil strata under consideration.

3. Please provide an opinion on the findings and recommendations in the rapid drawdown technical memorandum.

   Comments have been provided in the BOSC report that is part of this document.

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

   These were discussed at the meeting and are further discussed in the BOSC report. We anticipate additional discussion once BCI takes into consideration the BOSC comments and makes final recommendations concerning embankment fills.

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

The BOSC Report, which follows, fully addresses each concern.
Topics Discussed

I. Introductions

II. Overview Design Background

III. Overview of Preferred Alternative

A. How preferred alternative was developed
   1. Factors of technical feasibility and then affordability were examined. Namely:
      (1) Identify measures that were technically feasible.
      (2) Minimize impacts to existing landowners
      (3) Maximum benefits to the City.

B. A Value Engineering (VE) Report will be developed for segments B and F that will examine cost sharing possibilities and cost savings for the City can be achieved.

C. The BOSC stated that the 3 R’s (robust, redundant, and resilient) need to be addressed in the design throughout design, not just at the end.
   1. The Basis of Design Reports (BODRs) submitted throughout design and the VE report will incorporate the 3 R’s analysis.

D. The BOSC would like the design team to seriously consider relief wells at a few locations.
1. Relief wells were initially ruled out because of the geotechnical team's low level of confidence in their technical feasibility along our project reach, their relative rarity in the Sacramento area and the cost to maintain them.

2. An appendix evaluating the use of relief wells will be included in a geotechnical evaluation to be submitted to the BOSC in April 2012.

E. The BOSC questioned whether there is a need to incorporate a waterside inspection road for a setback levee.

1. It is a CVFPB requirement to have access on the waterside as well as the landside.

2. The BOSC noted that surfacing the road with gravel or base material may be an unnecessary expense.

3. Maintenance of the waterside road would be part of O&M.

F. There was a discussion regarding underseepage and the impacts on groundwater at borrow sites.

1. There was a concern about borrow sites being too close to the levee. Borrow areas should be 100 ft. away –landside of the levee.

   a) The design team would like to possibly use a waterside borrow area 50 ft. away from the levee.

   b) Borrow sites closer to the levee are favorable due to the economy of transporting materials a short distance to the placement area.

2. The BOSC and USACE want to look at groundwater effects on borrow sites.

   a) The Corps noted that groundwater was high at the north end of the project site where they are upgrading the levee.

      1. Similar problems along our project reach could result in shallow borrow areas or borrow areas that cover more area and less depth.

   3. The design team has been and will continue to investigate potential impacts to seepage caused by adjacent borrow activities.

IV. Developing Project Basis of Design

A. The BOSC recommended that borrow estimates should be developed using different assumptions regarding groundwater

B. The three Rs discussion should also be included.
V. Geomorphic Presentation

A. The BOSC recommended a Wind/Wave analysis should be made on soft erosion control measures.

B. The BOSC also noted that sea level rise should be included in modeling evaluations.

C. The design team should also look into what type of material the current rip rap on the levee sits on, specifically whether there is a filter layer to prevent loss of material through the rip rap.

D. The rip rap gradation template also to be re-evaluated.

E. Bee’s Lake’s hydraulic communication with the river.
   1. The new levee may be affected by Bee’s Lake water surface elevations.
   2. To address the possible contamination issue from Bee’s Lake to river, a phase 2 investigation will be completed for Bees Lake.
   3. Marina access roads could be built to a lower standard than a levee but a breach of that embankment on the upstream side of Bees Lake could cause a weir flow situation, and associated turbulent scour along the toe of a setback levee.
   4. Inundation times on the levee face near Bee’s Lake should be evaluated.
      a) What is the inundation criterion?
      b) BCI will evaluate rapid drawdown assuming 200-yr water surface elevations in Bees Lake. Is modified lower elevation warranted for this analysis?

IV. Geotechnical Presentation

A. Soil strength parameters table to be revised.

B. Soil types table to be revised to eliminate CL and include CH.

C. BCI to revise model and cross-sections.
   1. Cross Sections to be revised as more data is received.
   2. Models and cross-sections to be compared to boring logs for verification of the model.

D. BCI to also review Tables 6 and 7 for thickness of berm questions and factor of safety issues.
E. Consider Duncan & Wright’s method for assigning soil strength and stability especially on the waterside of the levee.

F. BOSC will provide where they think additional borings should be made.

1. BCI to send full-size hard copies to BOSC and electronic copies to Mary Perlea with the Corps of stick log profile sheets and plan view sheets showing location of explorations.

G. The BOSC was asked what they considered was a reasonable frequency for confirming the presence of a cutoff wall finish layer.

1. The BOSC recommended one every 100 ft. with much closer intervals for in-situ deep mix walls. For a 40 ft. deep wall (found in Segment A), 200 to 500 ft. may suffice.

H. There was a discussion regarding the liquid limit allowances for borrow material.

1. The BOSC recommended that a document that details what action should be implemented if the soil taken from the borrow site does not meet the requirement.

2. Language will be added to the DDR so the regulatory agencies are aware that this issue is being addressed.

3. BCI to develop a TM discussing typical zone sections, how different material types may be used in those sections, and criteria for acceptable levels of deviation from specified material properties.

VII. Quality Management Plan (QMP) Presentation

A. The QMP currently is being finalized with final completion by March.

B. The BOSC made these recommendations to the Comment/Response spreadsheet to be used.

1. Add a column that identifies the organization of the reviewer.

2. Be sure to provide detail in responses i.e., if you “agree”, explain what action was taken as a result of the responder agreeing. Example: For spelling errors, responder notes “Agree. The sentence was revised to correct the spelling error.”

3. Provide another column that verifies that the response was back-checked and the action noted in the response was completed.

C. The BOSC was also asked to provide comments to the Technical Approach Memorandum.
VIII. Site Civil Design Challenges

A. The BOSC noted that rapid draw down should not cause levee failure if the factor of safety has been met.

B. The design team clarified that failure planes that do not meet the required FS and that penetrated the template are assumed to be a problem — i.e. that the material should be removed and replaced with engineered fill to meet the FS or that the levee template should be shifted landward so that the noted failure plane does not penetrate the template.

C. The BOSC recommended that a modified crest width (less than 20 ft.) be considered when defining the template in segment A.

D. Discussion of the location of the public roadway near the berm in Segment B.
   1. Currently, the public road would be designed at the toe of the berm and a utility corridor would be placed landward of the road. South River Road would not have utilities (i.e. light poles) since County road standards, which apply, do not require them.
   2. The BOSC raised concerns about saturated roadway foundation conditions during high flow events were the roadway location to stay at the toe of seepage berms.
   3. The road could be placed on the berm and meet county road standards but not those required of a larger thoroughfare, such as Village Parkway, which would have lighting and curb and gutter drainage requirement. Such features would in turn require a substantial overbuild of the road section to keep the subsurface portion of these utilities outside of the seepage berm template.
   4. Placing the road at the top of the levee was not considered because of the Kleinfelder report stating that the levee was potentially seismically deficient, access issues, and that City prefers to have the road away from the river.
   5. The Corps noted that roadway foundations should not fall within the freeboard zone of the levee.

IX. Groundwater Presentation

A. There was a better correlation with groundwater and river stage in non-clay soils than in clay soils. The correlation was muted when in clay (Segment E).

B. Though this year has been a dry year, we have some data from the wet years of 2005 and 2011.

C. The BOSC would like to have a 90% confidence curve in correlation to water stage.
D. DWR has 18 piezometers in the West Sacramento area with a pair in Segments G and a pair between Segments C and D. One piezometer on the waterside shoulder of the levee and one at the toe.

1. Glenn Browning will do a data exchange with Steven Sunding from DWR to obtain information on those piezometers (SGDER and PGDER1)

E. LSCE plans on sending a survey to property owners about their wells and also to ask permission to be allowed on their property to examine their wells. This effort should be coordinated through the project focus group initially and, pending its approval, through Ken Jameson at the City.

X. Other Items

A. BOSC will send out a close-out 3 R letter after submittal of a construction documentation report for the CHP Academy and the Rivers EIP sites so they can review and verify construction.

B. TMs to be submitted between the 15% and 65% deliverables on key design challenges.

C. The BOSC advised that the GBODR should be final by the 90% submittal.

D. Project Milestones:

1. 65% by the end of July 2012
2. 90% by November 2012
3. 100% by February 2013
4. Bid Set by March 2013

E. Next meetings with the BOSC

1. Formal meeting week of May 14, 2012
2. Working teleconference in June 2012.
3. Formal meeting in August 2012 after the submittal of the 65% Plans and DDR.

F. Project construction will be done in phases over three years. The design team will provide phasing constraint within which the contractor(s) will be free to propose construction ‘sub-phases’.

1. The City prefers that Segment C be constructed first since it there will be minimal difficulty acquiring real estate for the project in this area.
Attachment #5

BOSC Comments on Documents provided for Review:
February 21-22, 2012 BOSC Meeting

Each document is reviewed below. In addition to the comments provided, questions and recommendations for modifications to the text or analyses are underlined for emphasis.

Memorandum, Approximate Rip Rap Coverage
cbec eco engineering, dated January 18, 2012

In regard to question 1, page 1, it was answered that the guidance for the required upper limit of the riprap was from NCHRP 568, which calls for the design water surface elevation (Q200) plus 2 feet. The CA Draft Urban Design Levee Criteria (ULDC, Nov. 15, 2011) specifies the minimum top of levee (MTOL) as the higher of the Design Water Surface Elevation (DWSE), or the median 200-year water surface elevation plus three feet, or the median 200-year water surface elevation plus computed wind setup and wave runup. We recommend that the riprap extend to the MTOL, as defined by the CA ULDC.

On page 1, the Memo stated in a general way that the riprap rocks often broke when dropped, indicating substandard material. A description of the type of stone was not presented. Before the existing riprap is assumed to be re-usable, an opinion by the Corps should be obtained in regards to the acceptable material specifications of the riprap, such as hardness, density, etc.

West Sacramento/Southport EIP: Task Order 3: Revetment Condition Assessment
cbec eco engineering, dated January 16, 2012

General. The report did not mention gathering of any as-builts or design documents of the riprap or any maintenance activities at the sites. This information would be valuable when designing future riprap because the successes and problem locations can be compared to the current design.

Section 2.0 Field Methods

Subsection 2.1 Rip Rap Delineation

The toe of riprap was identified by probing with an 8 feet steel soil probe. For subsequent riprap delineation studies, it is suggested that other methods be used, such as sonic. At a minimum, the probe should be 10 feet long.

Subsection 2.2 Transect Survey

The Memo states “It is assumed that for such a recent project, the data regarding rip rap characteristics will be obtain from the agency responsible for its construction.” This data should be obtained.
Page 11, Table 3-2. For Transect F2, the HIGH sample showed a density of 76 psf. This should have been reported as an outlier and highlighted as such in the table and not used to obtain the average for the transect. Also, please note that the Corps does not allow a riprap density less than 150 psf unless the layer thickness is increased.

Subsection 3.2 Theoretical Evaluations of Riprap Revetment Requirements

Page 12. For determining the wetted width of the channel, make sure to use only the distance between the top of banks of the channel, not the entire wetted width. Also, note that the NCHRP relation between \( D_{50} \) and \( D_{30} \) is based upon an assumed standard gradation. A more accurate relation is:

\[
D_{50} = D_{30} \left( \frac{D_{85}}{D_{15}} \right)^{1/3}.
\]

Page 14, Table 3-3. Please cite the thickness of the riprap layer as thickness, not depth.

Section 4 Explanation of Findings

Subsection 4.1 Particle Characteristics

4.1.1 Size

Please change title of 4.1.1 from size to gradation.

4.1.3 Depth

Please change title of 4.1.3 from Depth to Thickness.

Section 5 Discussion

Note that the assessment of potential riprap failure is based upon the Corps’ methodology which is based upon the shear stress of water moving generally parallel to the bank. As noted in the report, some of the damage to the existing riprap was due to wave action. As design goes further, make sure to include this in the design.

Section 6 References

Please update the first reference to DWR, Draft Urban Levee Design Criteria, November 15, 2011
Draft Technical Approach Memorandum  
HDR, dated December 19, 2011, Revision 5

Section 3.0 Federal and State Guidance Documents

Item 28 - “Interim Levee Design Criteria for Urban and Urbanizing Areas in the Sacramento-San Joaquin Valley”, the current version should be referenced.

Item 31 – The reference to TM 3-424 should be removed. This document is referenced in item 1.

Section 5.0 Hydraulics and Hydrology

Subsection 5.1 Design Water Surface

Subsection 5.1.1 Design Water Elevations

On Page 6, it should be pointed out that the 1-D hydraulic model is an unsteady flow model. Also, the hydraulic model developed by cbec for the lower flood events should be stated as 1 or 2-D and steady or unsteady model.

Subsection 5.1.3 Levee Height

Shouldn’t settlement of new levee construction be added?

Subsection 5.1.4 Wind and Boat Wave Runup and Setup


Subsection 5.1.5 Bridge Freeboard

The FEMA requirement for the freeboard upstream and downstream of a bridge for a 100 year flood was cited (CFR 65.10). However, there are no recommendations of this type for the Southport bridges. The Board recommends the additional freeboard near bridges per CFR 65.10 for the 200 year flood in addition to the 3 feet of freeboard above the DWSE.

Subsection 5.2 Erosion Evaluation and Protection Design

Subsection 5.2.4 Wind and Boat Waves


Subsection 5.3 Design Criteria

Subsection 5.3.1.3 Top, End, and Toe Protection

What methods will be used for bend scour and maximum scour? Will the scour depths be applied to the thalweg and assumed to move to the worst location?
Section 7.0 Geotechnical Data Collection

Subsection 7.2 Explorations and Testing

The guidance reference to TM 3-424 probably should reference EM 1110-2-1913 and ETL 1110-2-569. Additionally, is the reference ER 1110-2-1807 the correct reference for drilling through levees? We believe this reference should be ER 1110-1-1807.

Section 8.0 Geotechnical Design Criteria and Analysis

The DWR reference to their Levee Design Criteria should reference the current version.

Subsection 8.3 Geotechnical Design Criteria

Subsection 8.3.3 Static Slope Stability

There is no discussion of the End of Construction Stability case. It is suggested that the following language be added from EM 1110-2-1913, Design and Construction of Levees, Chapter 6, Slope Design and Settlement, Section 6-5, page 6-3. Conditions Requiring Analysis:

“Case I - End of construction. This case represents undrained conditions for impervious embankment and foundation soils; i.e., excess pore water pressure is present because the soil has not had time to drain since being loaded. Results from laboratory Q (unconsolidated-undrained) tests are applicable to fine-grained soils loaded under this condition while results of S (consolidated-drained) tests can be used for pervious soils that drain fast enough during loading so that no excess pore water pressure is present at the end of construction. The end of construction condition is applicable to both the riverside and landside slopes.”

Subsection 8.3.4 Slope Stability Analysis

The river stages proposed for rapid drawdown appears severe. There is absolutely no possibility of the levee, constructed of Type I material, becoming saturated to the Design Water Surface Elevation (DWSE).

Subsection 8.3.7 Levee Foundation Settlement

Is EM 1110-2-1904 the correct reference? We believe that it should be EM 1110-1-1904.

Section 9 Levee Remediation Techniques

Subsection 9.2 Cutoff Walls

Why is a 36 inch minimum thickness required?

Subsection 9.3 Seepage Berms

Under CESPK SOP 3, the requirement to place a drainage layer under a semi-pervious berm is not necessary as long as USACE requirements for semi-pervious berms are met. Additionally, the use of a drainage layer under these berms can pose a significant public life safety issue due to the potential for piping if proper filters are not included.
River and Levee Erosion Evaluation

HDR, January 2012

Section 4.0 Erosion Potential on Upper Bank

Subsection 4.1 Current Conditions

The sentence, “No erosion was identified in Segments A and B, but the length of those segments has a continuous presence of riprap” does not appear to agree with the next sentence. “Furthermore, some of the riprap in the Southport Project Reach is failing from erosion, or is not effectively protecting against erosion.”

Subsection 4.2 Mechanisms of Upper Bank Erosion

The various types of erosion have been described but the types of observed erosion have not been presented. Is there information on this on a reach by reach basis?

Subsection 4.3 Estimates of Upper Bank Erosion Distances

All of the erosion rates are based upon 2 observation times and then assumed to be a linear extrapolation to the project life of 50 years. The BOSC agrees that, although erosion rates have a wide range, there is significant potential for bank erosion over the 50 year life and it is prudent to plan for this.

Subsection 4.4 Locations of Potential Upper Bank Erosion

A 2-D horizontal model was used to determine upper bank erosion potential by comparing computed velocities to the allowable velocity of the material on the bank. At bends, the velocities and the resulting shear stresses are due to 3-D phenomenon which can be underestimated by a 2-D model. For future design, the results of the 2-D model should be adjusted higher at bends.

Section 5.0 Erosion Potential at Bank Toe

Subsection 5.1 Long-Term Channel Bed Lowering

It is stated that the rate of channel incision has decreased substantially. Why is this and is the phenomenon that causes this decrease expected to continue in the future?

Subsection 5.2 Existing Scour Holes

It appears that all of the significant scour holes are at the outside of bends. It is suggested that the depth of these scour holes be compared to empirical methods, even if not affecting the levees, and if appropriate, use for the design where the levees are affected.
Subsection 5.3 Maximum Scour Depths

Regime methods were used in the PIR to determine the maximum scour depths due to a 200 year flood and the results were no greater than the maximum observed depths. It is unclear if the computed scour depths included bend scour.

Subsection 5.4 Locations of Potential Bank Toe Erosion

The text states: “The velocities were compared to the maximum allowable velocity, above which shear stress erosion might be expected. The allowable velocity was selected based on bank soil material because, as discussed previously, it could not be determined whether the existing riprap extends to the channel toe. Also, this assumption would represent conditions where there is no riprap or the hypothetical condition where riprap becomes unmaintained. Bank material along the Southport Project reach typically consists of a layer of fine grained silt underlain by up to 100 feet of sand and gravel, with interbedded silty sand and clayey sand layers (nhc, February 2008).” The report references soils data from nhc, dated February 2008. Significant new data were provided by Blackburn Consulting, Inc. (BCI) on November 15, 2011 (BCI November 2011 TM) in a report titled “Technical Memorandum, Geotechnical Design Document, Borrow Site Evaluation, Soil Sampling for Scour Analysis, Seismic Ground Motion.” Why were these data not used in this analysis? The Blackburn report was not even referenced in the report.

As pointed out in the Section 4.4 comment, adjustments to the 2-D model results should be made at bends.

Section 6 Meander Migration Potential

Subsection 6.2 Application of Bend Migration Data

Table 1 - The estimated meander bend migration potentials listed in this table appear excessive. From these values, it would appear that none of the proposed levee setbacks will be adequate without additional river bank stabilization. Additionally, Reaches A and B, which earlier in the report indicated no erosion, is listed here to have the potential of 360 feet. This apparent contradiction needs to be clarified.

Section 7 Summary of Bank Erosion Potential and Preliminary Recommendations

All of the reaches discussed here require riprap to be maintained in order to prevent erosion. Therefore it would appear that bank stabilization is required for all alternatives; is this correct?

Low, medium and high estimates should be made for the erosion rates. The estimated parameters appear to be biased toward the high erosion rates.

Section 8 References

The DWR reference to their Draft Levee Design Criteria should reference the current version.
Technical Memorandum, Geotechnical Design Document, Borrow Site Evaluation, Soil Sampling for Scour Analysis, Seismic Ground Motion
BCI, November 15, 2011

Section 2.0 Continued Borrow Site Evaluation

The text states “BCI determined the ranking based on haul route distance and a confidence level that subsurface soils above groundwater will meet levee fill criteria defined as Type 1 or Type 2 Levee Fill shown in Table 1.” We believe BCI should determine these types of material to a prescribed depth regardless of the depth of the water table. Also the presence of other material within these areas should be determined.

Subsection 2.1 Subsurface Exploration and Subsection 2.2 Laboratory Testing

The initial borrow site evaluation was completed using borings, test pits and laboratory data from other sources since BCI had not as yet drilled any new test borings at the potential borrow sites. These data were reported in a Technical Memorandum, Geotechnical Design Considerations, dated January 25, 2011 (BCI January 2011 TM). In this TM, none of these initial data were included. Interpretation of the data is the responsibility of BCI, but it seems inappropriate to ignore the previous data. It is recommended that the BCI November 2011 TM be revised to include: 1) borings drilled and test pits excavated by others; 2) laboratory testing performed by others; 3) cross sections illustrating the entire data set (see Exhibits G-4 through G-22 included in the BCI January 2011 TM). It is not possible to properly evaluate the conclusions drawn unless all of the data are presented. Note that BCI included the borings and CPT soundings by others along the levee alignment and the borrow areas. This information should be treated the same with respect to documentation.

Subsection 2.5 Recommendations

A recommendation on page 6 states: “Consider lime treatment of Type 2A soils within Riverpark 2, Liberty 2 and Yarbrough in order to achieve acceptable fill type requirements.” It appears, based on current information, that if Yarbrough fill was used, lime treatment would be required. However, on the other sites, BCI should determine the percentage above the LL requirements and then if relatively low, require blending.

On page 7, it states: “Further evaluations should consider expansion of the investigation areas laterally beyond the current borrow site boundaries to consider the nondeveloped land in-between the areas evaluated in this TM.” Consideration should be given to reducing the number of borrow areas and areal extent of the borrow areas and to allowing excavation below the water table. This will require the contractor to lower the groundwater table by a method they choose and will greatly reduce area of disturbance.

Section 3.0 Existing Levee Sampling and Testing for Scour Analysis

As noted above, these data should be used in the HDR erosion evaluation.
Section 4.0 Site Seismicity

Subsection 4.1 Site Specific Ground Motion

The text states: “A weighted average based on contribution results in M equal to 6.74. We select an applicable M equal to 6.7 for use in project analysis.”

“We note that the deaggregation also shows some contribution from the Cascadia Subduction Zone at a distance of over 174 miles (280 km) and M ranging from 8.8 to 9.2. The contribution is low (less than 2%) and, therefore, not a likely event at the 200-year return period. However, BCI will consider this distant event and large M value in evaluation of the liquefaction hazard.”

These statements seem incompatible. The liquefaction analyses require an earthquake magnitude, M. A probabilistic analysis was performed to estimate that M so it seems that 6.7 is the correct M to use in the liquefaction analyses. Why would an M equal to 8.8 to 9.0 be used? This M earthquake will surely have a negative and unnecessary impact on the liquefaction analyses. Why would this large earthquake be considered independent of the PSHA since its small contribution was already included in the analysis?

Section 6.0 Tables

Table 1 - Consideration should be given to define Type 2 material’s LL to be material that has a LL between 45 and 60.

Table 1 thru Table 8 - It is somewhat confusing when materials are checked as meeting Type 1 and Type 2. It would appear that the designers would want to determine how much Type 1 material was available and required; also, how much Type 2 material was available and could be used. Additionally, the material not meeting Type 1 and 2 requirements would be excellent berm material or could be blended with high LL material to make Type 1 or 2 material.

Table 8 - Atterberg Limits are a relative cheap test. Why weren’t more performed in this area to better determine material types and quantities?

Appendix A Laboratory Test Results

All the results shown in this appendix are not shown on the boring logs. Please include all the appropriate data.
Draft Rapid Drawdown Technical Memorandum, Rapid Drawdown Analysis  
BCI, February 3, 2012

Section 3.0 Soil Drainage Properties

Subsection 3.1 Native Clays and Plastic Silts

The use of the Duncan and Wright equation, shown on page 3, for evaluation of whether a particular soil will act as though it is drained or undrained, appears reasonable.

We agree that soils classified as clay should be evaluated as undrained as long as they have reached 100% saturation. This assumes that the flooding event will remain in place for sufficient time to cause the clay soils in the levee to become saturated.

The text states on page 3: “For consistency and to roughly simulate expected consolidation loads during drawdown, we evaluated $c_v$ values for surcharge loads of 500, 1,000 psf and 1,100 psf where data was available.” It would be helpful in the evaluation of the approach if BCI would expand on how the surcharge loads were calculated.

The text on page 3 states: “Therefore, judgment should be used to determine if plastic silts should be modeled drained or undrained based on the plasticity of the soil, sand content and conservatism of the other input parameters.” What are the values of the parameters to be used to make this determination? How can the clays discussed in this section become saturated during a flood?

Subsection 3.2 Sand, Silty Sand and Non-Plastic Sandy Silt

We agree that soils with $k_h \geq 10^{-4}$ cm/sec should be considered drained for analysis purposes. This criterion should be used as a check on the Duncan and Wright approach.

Subsection 3.3 Compacted Clayey Levee Fill

There is the possibility that the embankment soils may not become saturated to the full depth of the rapid drawdown potential failure circles because the flood stage only lasts for about 5 to 15 days as noted on page 4. It has been demonstrated that the permeability values are very low. Thus, we recommend that the rapid drawdown analysis be performed using two water surfaces to evaluate the impact: 1) assume the phreatic surface has been established through the embankment (as BCI has chosen to do in their analyses) reported, and 2) assume the water surface is at the face of the embankment and that no penetration of the wetting front has occurred. The results of the two analyses can then be compared and a decision made as to how to interpret the analyses. The clayey soils that are located within the river bank should be considered undrained for purposes of rapid drawdown analysis.

Section 4.0 Past Rapid Drawdown Levee Performance

The text states on page 4:

“The waterside slides discussed above may be a result of rapid drawdown and/or undercutting caused by scour.”
During the February 21 - 22, 2012 BOSC meeting, Mr. Ken Ruzich indicated that he was not aware that any of the slides were related to drawdown of floodwaters. He noted that it was his opinion that they were related to scour. This discrepancy should be clarified.

Section 5.0 Conclusions

To start the drawdown elevation at the 200-year design water surface is very, very conservative. We do not agree with the conclusion that “an average F.S. of 1.1 is reasonable for rapid drawdown analysis in conjunction with the 20-foot drawdown”. With the extreme assumptions discussed in this report, a more appropriate F.S. of 1.0 should be used.
Section 1.0 Introduction

“The Revised Draft TMA contain edits based on HDR’s initial internal review of the December 6, 2011 Draft TMA. BCI prepared this Revised Draft TMA to our ‘Geotechnical Design Document – Southport Early Implementation Project – Project Preliminary Design,’ dated November 14, 2011 (TMPPD). This Revised Draft TMA contains additional information and analysis for Final Preliminary Design not contained in the TMPPD.”

We were not provided a copy of the November 14, 2011 document for review. It appears that the November 14, 2011 document has been updated in addition to being modified based on the HDR comments to produce this TMA. The number and naming of these memoranda is somewhat confusing.

Section 2.0 Subsurface Exploration and Laboratory Testing

Subsection 2.1 Subsurface Explorations

To date, BCI has completed the following explorations as noted in this TMA:

- Test borings – 21
- Sonic borings – 8
- CPT soundings – 18
- Test pits – 47

The test borings are labeled BCI-11-B1 through -B5, BCI-11-11 through -B25 and BCI-11-B29. The sonic borings were designated BCI-11-B6 through -B10 and BCI-11-B26 through -B28. The CPT soundings were labeled BCI-11-CPT1 through BCI-11-CPT18 according to Appendix A. The title page of Appendix A should be corrected to reflect 18 CPT soundings, not 17 as presently shown.

Of note was the refusal of CPT probes BCI-11-CPT14, -CPT15 and -CPT16 at depths of approximately 83 feet, 58 feet, and 71 feet, respectively. Sonic boring BCI-11-B9 refused at 86 feet in gravel/cobbles.

Subsection 2.2 Laboratory Testing
Review of the laboratory testing data in the BCI January 2012 TMA indicates that samples were tested from all borings except Sonic Borings BCI-11-B6 through –B10. Why were there no tests performed on these disturbed samples: should tests have been performed?

The TMA noted that potential borrow sites designated Riverpark 1 and 2; Liberty 1, 2 and 3; Newport 1 and 2; and Yarbrough were to be evaluated. Appendix B only reports laboratory data for Liberty 1 and 2, Riverpark 1 and Yarbrough. Were the data from previous studies used in the evaluation or has the testing not been completed?

Section 3.0 Historical Levee Performance

Subsection 3.3 Identified Levee Distress

Why was the levee performance during the 1995 flood not presented?

Section 4.0 Subsurface Conditions

Subsection 4.1 Segment A

Segment A extends from Stations 0+00 to 43+00, according to BCI July 2011TM – FPD. Exhibit G-6 of the TMA contains boring data through Station 46+00. The last 300 feet of data on Exhibit G-6 should be included on Exhibit G-7 for Segment B. These data are excluded on Exhibit G-7.

The text states on page 5 that “[t]he levee is generally underlain by about 5 to 18 feet of low plasticity soft to medium stiff silt/sandy silt and clay/sandy clay.” This layer is underlain by “stiff to hard, medium to high plasticity clay and silt to depths of 24 to 47 feet” which is “underlain by dense sand and sand w/silt (with some zones of silty sand) to depths of at least 53 to 64 feet.” The upper 5 to 18 feet appear to represent a blanket layer. The subsurface description of Segment A appears consistent with the test boring and CPT sounding data.

The TMA also states on page 5 that “Figures K-14A, 14B, 15A and 15B of the HEM Study support the subsurface descriptions presented above---.” The helicopter electromagnetic (HEM) geophysical surveys give a relative indication of the soil types in this riverine lenticular deposited alluvium. The upper soils of most interest are suggested to be silts to sands (high resistivity – red and yellow). The test borings, CPT soundings and laboratory data indicate that these soils are more fine grained than suggested by the geophysical data. The HEM data should be calibrated to the soil types at Southport or an explanation should be provided to explain the relative nature of these results. In this segment, the HEM data indicate the clay layer (low resistivity – blue) exists below about 80 feet.

The TMA states that “Explorations within the mapped crevasse splay area do not indicate significantly different conditions than those outside of the crevasse splay.” Consider adding a sentence that explains this. For example: The exact locations of these are sometimes hard to locate. However, knowing they exist requires the designer to design for their existence. This comment also applies to other segments.

Subsection 4.2 Segment B

Segment B extends from Stations 43+00 to 100+00 according to BCI TM – FPD July 2011. The text of TMA states on page 6 that the subsurface conditions are similar to Segment A from Station 43+00 to 47+00. Beyond Station 47+00 to Station 100+00 “the upper 5 to 10 feet of soil---consists
predominantly of medium plasticity silt/clay with isolated zones of stiff, high plasticity clay and soft to medium stiff, low plasticity sandy silt/sandy clay.” The thickness of this layer appears to be mischaracterized as it appears to be much thicker than stated in the report. The description goes on to state that this surficial layer is “---generally underlain by 25 to 45 feet of medium dense silty sand to firm, generally non-plastic silt/sandy silt.” It appears more reasonable, based upon review of the boring logs, to combine these two layers as they are predominantly fine grained material. At a minimum, the combined layer should extend to El 0 so it is at least 20 feet thick and, in some areas, up to 40 feet thick (El -20).

The TMA does point out correctly that one boring near Station 53+00, which appears to be Boring S-36 located at about Station 56+00, has sand in the upper 5 to 10 feet. A second Boring S-9 located at about Station 58+00 also contains an upper sand layer. Both of these borings are located along a road. There is a good possibility that these soils may be fill associated with pipeline work. This should be investigated before drawing conclusions about the significance of these soils. Note that both borings indicate lean clay to about El -20 below the surficial sand layer.

The TMA states on page 6 that “Figures K-13A, 13B, 14A and 14B of the HEM Study support the subsurface descriptions presented above, particularly the similarity to Segment A up to Sta. 47+00, and the deepening clay layer [>120feet] below the sand/gravel layers north of Sta. 47+00.” This is a more reasonable interpretation then for Segment A. It could also be interpreted that the deep clay layer dips below 150 feet in depth at the end of Segment B.

Subsection 4.3 Segment C

Segment C extends from Stations 100+00 to 165+00 (existing levee stationing) according to the BCI TM – FPD July 2011. The stationing is somewhat confusing in that Exhibits G-6 through G-12 uses the existing levee stationing but Exhibits G-XA through G-XH, handed out during the BOSC meeting of February 2012, plot data based on the new levee stationing. For purposes of this review, we have used the existing levee stationing. The TMA notes on page 7 that only the portion up to Station 158+00 is discussed under Segment C and that the northern portion from Station 158+00 to 165+00 is similar to Segment D and is discussed under that segment. Review of the segment borings and CPT soundings suggests that the boundary of the northern portion could be extended south at least 400 feet to about Station 154+00 such that the portion similar to Segment D extends from Station 154+00 to Station 165+00. It is suggested that BCI review this boundary location.

The TMA indicates on page 7 that the subsurface conditions in the upper 10 feet of this segment (Station 100+00 to 154+00) consists of “---variable layers of loose silty sand and soft to stiff, low plasticity sandy silt/sandy clay.” Below the surficial layer, the soils to a depth of 25 to 40 feet typically consisted of “---loose silty sand to soft to stiff (generally non-plastic) sandy silt with small zones of low plasticity soft to stiff clay/silt.” As in Segment B, there appears to be more fine grained soils than suggested by the TMA, but the sands appear to contain less fines and make up a larger portion of the subsurface section than in Segment B. Below this layer is “---a layer of dense gravel that ranges in thickness from several feet to up to 35 feet--.” This appears to be a mischaracterization of this layer as it appears to contain a significant portion of poorly graded sands. This difference will not significantly change the permeability characteristics of this stratum.

The TMA states that “K-12A, 12B, 13A and 13B of the HEM Study support the subsurface descriptions presented above.” The geophysical survey suggests that this segment contains considerably more granular soils. The deep clay layer dips below 150 feet in depth in the central portion of the segment.
Once again, the geophysical results appear to suggest that the subsurface is more granular than the test boring data indicates.

**Subsection 4.4 Segment D**

Segment D extends from Stations 165+00 to 183+00 (existing levee stationing) according to the BCI TM – FPD July 2011. As noted above, the portion of Segment C from Station 158+00 (suggested to be 154+00) to 165+00 (existing levee stationing) is similar to Segment D and thus is included in this Segment D discussion.

The TMA indicates on page 7 that the subsurface conditions in “the upper the upper 7 to 10 feet of soil--generally consists of variable layers of loose to medium dense silty sand and soft to medium stiff sandy silt. This layer is underlain by 15 to 25 feet of medium stiff to stiff, medium to high plasticity clay.” The description below this level indicates the soils are silty sands and sandy silts “---with some relatively clean sand lenses---to the 130-foot depth explored.”

The report indicates that the HEM Study generally supports the subsurface descriptions presented above and this appears to be a reasonable assessment.

**Subsection 4.5 Segment E**

Segment E extends from Stations 183+00 to 214+00 (existing levee stationing) according to the BCI TM – FPD July 2011. The TMA indicates on page 8 that the subsurface conditions south of Station 197+00 (existing levee stationing) are similar to Segment D. Review of the geologic profile on Exhibit G-10 suggests that the transition should be located about 400 feet farther south at Station 193+00.

The TMA indicates on page 8 that the subsurface conditions south of Station 197+00 in the upper 7 to 10 feet consist “---of variable layers of loose to medium dense silty sand and soft to medium stiff sandy silt.” This layer is “---underlain by 15 to 25 feet of medium stiff to stiff, medium to high plasticity clay.” The description below this level indicates the soils are silty sands and sandy silts “---with some relatively clean sand lenses---to the 100-foot depth explored.” North of Station 197+00, the upper 11 to 15 feet “---of soil consists of variable layers of soft to firm low plasticity clay/silt and loose sand/silty sand, which is underlain predominantly by loose to medium dense silty sand and relatively clean sand to depths of about 42 to 51 feet.” The difference in the two portions of this segment appears to be that the clay soils below the surfical soils are missing north of Station 193+00.

The report indicates that the HEM Study generally supports the transition at about Station 197+00 and this is a reasonable assessment.

**Subsection 4.6 Segment F**

Segment F extends from Stations 214+00 to 273+00 (existing levee stationing) according to the BCI TM – FPD July 2011. The TMA indicates on page 9 that the subsurface conditions in “the upper 20 to 30 feet of soil---generally consists of soft to medium stiff low plasticity to non-plastic silt with lenses of silty sand. At some locations, there are layers of medium stiff to stiff low to medium plasticity clay and silt in the upper 3 to 5 feet. Below 20 to 30 feet, the explorations indicate medium dense to dense sand with silt and gravel to about 90 feet depth. Dense gravel and sand generally extend from 90 feet to a depth of at least 100 to 135 feet, underlain by stiff clay/silt.”
The report indicates that the HEM Study generally supports the deep sand and gravel at depth in this segment.

Section 4.7 Segment G

Segment G extends from Stations 273+00 to 296+00 (existing levee stationing) according to the BCI TM – FPD July 2011. The TMA indicates on page 9 that the subsurface conditions in “the upper 10 to 15 feet of soil---consists predominantly of low plasticity clay/silt with some zones of silty sand, which is underlain by silty sand to depths of about 20 to 30 feet. Below this layer, relatively clean sand extends to depths of about 45 to 60 feet, underlain by a 20- to 35-foot-thick layer of dense, coarse gravel.”

The report indicates that the HEM Study generally supports the subsurface description indicated by the boring and CPT sounding data.

Section 5.0 Geotechnical Analyses

BCI updated the five original cross sections and added three new cross sections in the TMA. These included: Station 19+00 (new), Station 79+00, Station 122+00, Station 167+00, Station 197+35 (new), Station 205+44 (new), Station 241+00, and Station 283+00.

Section 5.1 Underseepage and Slope Stability Analysis Model Parameters

No comments

Section 5.2 Underseepage Analyses

The permeability parameters selected for use in the seepage analyses were reviewed in Appendix F of the TMA and they appear appropriate. BCI used Seep/W Version 7.16 to perform the analyses.

Section 5.3 Slope Stability Analyses

The shear strength parameters selected for use in the stability analyses were reviewed in Appendix E of the TMA. The parameters shown in Table E-1 seem reasonable for the ML/CL/ML-CL/CH (PI>7) and ML (PI<7, note Table E-1 mistakenly indicates PI>7). During the February 2012 BOSC meeting, there was a significant amount of discussion about what parameters to use for the CH clays. The BCI parameters seem reasonable based on the test results achieved. It is true that CH clays tend to have lower \( \phi' \) values than CL and ML soils but in BCI’s defense, one of these CH samples had a liquid limit of 50 and \( \phi' = 33^\circ \) and the second a liquid limit of 63 and \( \phi' = 25^\circ \). Thus, it would seem appropriate to reduce the recommended \( \phi' = 30^\circ \) to \( \phi' = 28^\circ \).

The recommended undrained shear strength, \( s_u \), of 1000 psf from the Unconsolidated Undrained (UU) triaxial tests of the levee fill material appears low based on the test results. The values were \( s_u = 1533 \) and 1820 psf. This recommended value of \( s_u = 1000 \) psf should be reviewed.

Questions were also raised about the shear strength parameters used for the soil bentonite slurry wall. The BCI values of \( \varphi = 0^\circ \) and \( c' = 300 \) psf appears too high. Values of \( \varphi = 0^\circ \) and \( c' = 20 \) psf were used in Natomas. The shear strength of the slurry wall should be reduced.
The TMA states on page 13 that the rapid drawdown “---analysis used both effective and total shear strengths shown on Table E-1 in Appendix E as input---.” BCI used Slope/W Version 7.16 which does have the ability to perform three stage rapid drawdown analyses. The effective and total strength parameters c’ and $\phi'$ and c and $\phi$, respectively, from the Consolidated Undrained (CU) triaxial tests with pore pressure measurements, were used as input. The analyses require use of the modified total strength parameters d and $\psi$ which are calculated from the total strength parameters c and $\phi$ as recommended by Duncan et al. Review of Slope/W Version 7.16 documentation indicates that the program calculates these required parameters.

Section 5.4 Settlement Analyses Results

No comments

Section 5.5 Seismic Analysis

No comments

Section 6.0 Conclusions and Recommendations

Subsection 6.1 Existing Levees

Subsection 6.2.1 Segment A (Station 0+00 to 43+00) – Evaluation Cross-Section
Station 19+00

The TMA states on page 16 that “[a] strengthen-in-place levee mitigation alternative is proposed by the design team and WSAFCA for Segment A.” We concur with a strengthen-in-place levee mitigation consisting of an adjacent levee and shallow cutoff wall because the stability factors of safety are not met for steady state seepage or rapid drawdown. We do have several questions as noted below.

The discussion in Section 4.1, Segment A indicates that the soils are “silt/sandy silt and clay/sandy clay--[and]--clay and silt” from the ground surface to depths of 53 to 64 feet. The geologic profile of Exhibit G-XA, that was handed out at the BOSC meeting in February, indicates a layer labeled SM/ML between about Stations 15+00 and 25+00 and between about El 0 and 10 to 15 by a greenish yellow color. This color indicates silty sand (SM), not silts (ML), and appears to be a misrepresentation of the conditions. At the location of the geologic cross section, Station 19+00, this SM/ML layer appears to be 10 feet thick. The same layer on the geologic cross section, shown on Exhibit G-13, indicates the layer is only 5 feet thick. Interestingly, the note on this layer on the geologic cross section indicates that the layer contains less than 50 percent fines and is classified as SM, not SM/ML. Review of the borings indicates that only one boring along the Segment A centerline, KA-06-03, has a silty sand layer at this level with a fines content of 48 percent. The remaining 11 borings indicate silt (ML) or lean clay (CL) between El 0 and 20 (two indicate shallow surfical silty sand layers). This does not appear to be justification for depicting such a large area on Exhibit G-XA as SM material (greenish yellow shading) even if it is described as SM/ML. The area should be much smaller and be designated SM or should be deleted and included with the surrounding ML/CL area.

A general comment is in order. Throughout this project, BCI have defined the stratigraphic layers as either coarse grained (SP, SP-SM, or SM (GM)) or fine grained (ML, ML-CL, CL, CL-CH or CH) except in a few locations where fine grained and coarse grained soils are combined into one stratigraphic layer designated SM-ML, ML-SM or SC-CL. It is apparent from review of the geologic profiles that the
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first designation, such as SM in SM-ML, indicates that the layer consists of more silty sand than silt.  
The problem with these fine grained/coarse grain layers is that BCI has not defined the permeability or  
shear strength parameters for these fine grained/coarse grain layers (see Tables D-1 and E-1 of  
Appendices D and E, respectively). Thus, an interpretation must be made to obtain the parameters for  
each layer. The issues associated with the selection of shear strength and permeability parameters are  
discussed in subsequent comments for each segment.

The seepage analyses (Exhibits G-14, G-15, G-19 and G-20) indicate that the permeability value used  
for the silty sand/sandy silt layer (24) is that for silty sand and is 4 x 10^{-4} \text{cm/sec}, which is more than  
an order of magnitude greater than for silt, which is 2 x 10^{-5} \text{cm/sec}. The selection of this permeability value  
should be reviewed.

The end of construction stability cases (Exhibits G-24) include effective strength parameters for:

New levee fill (2), \( c' = 90 \text{ psf} \) and \( \phi' = 24^\circ \);

The natural clay (22), \( c' = 0 \text{ psf} \) and \( \phi' = 30^\circ \);

The silty sand/sandy silt layer (24), \( c' = 0 \text{ psf} \) and \( \phi' = 30^\circ \).

This is incorrect. The total strength parameters from the Unconsolidated Undrained (UU) triaxial tests  
should be used for the new levee fill (2). The recommended value of \( s_u = 1000 \text{ psf} \) appears low as  
noted previously. The recommended undrained shear strength, \( s_u = 500 \text{ psf} \) seems appropriate for  
natural clay (22). The recommended undrained shear strength for the silty sand/sandy silt layer (24)  
with a minimum of 48% fines is 500 psf.

The recommended shear strength parameters for the lean clay/silt layer (9) were CU undrained  
parameters: \( c = 240 \text{ psf} \) and \( \phi = 13^\circ \). The Corps states in EM 1110-2-1902 Appendix D, page D-7, that  
the use of CU undrained shear strength parameters for the end of construction case requires the  
following approach:

> “Data from Consolidated-Undrained shear tests can be used to estimate the undrained shear  
strength of saturated soils for use in analyses for end-of-construction stability. By  
reconsolidating specimens in the laboratory, it is possible to reduce some of the effects of sample  
disturbance. However, care must be used to avoid increasing the strength, and overestimating  
the undrained shear strength. When Consolidated-Undrained shear test procedures are used to  
estimate undrained shear strength the undrained shear strength is expressed as \( S_u = (\sigma_1 - \sigma_3)/2 \)  
and is related to the effective consolidation pressure. Two approaches may be used to do this.  
One approach is the SHANSEP approach suggested by Ladd and Foot (1974); the other is the  
“recompression” technique suggested by Bjerrum (1973).”

This is very complicated and not recommended. The undrained shear strength, \( s_u = 500 \text{ psf} \) seems a  
better approach for natural clay (22).

Considering the rapid drawdown stability cases (Exhibits G-18 and G-23), it appears that the shear  
strength parameters for the silt/sandy silt (24) were selected assuming the layer is free draining.  
This appears to be incorrect as the layer, as presently defined, consists of both silts and silty sands (minimum  
48% fines). This material should not be considered free draining and both the effective strength and
total strength parameters should be used in the analyses: \( c' = 0 \text{ psf} \) and \( \phi' = 30^\circ \) and \( c = 240 \text{ psf} \) and \( \phi = 13^\circ \). This may impact the results of the analysis.

The rapid drawdown analysis was performed on a mitigated 3H:1V waterside slope with river water initially at El 32. The drawdown was specified to El 13, which is the average winter low water level. The factor of safety was found to be 1.2. When the analysis was completed for the existing 2H:1V slope, the factor of safety was only 0.9. This appears reasonable at first glance since the slope was flattened. Two issues should be evaluated to verify that the analysis is correct as described below.

Firstly, the pore water pressures are higher in the mitigated slope because of the presence of the “impervious” embankment fill and the cutoff wall which would suggest that the factor of safety should be reduced. It is suggested that the initial analysis be redone with a 3H:1V slope to better understand the impact of the slope flattening. If the resulting factor of safety is greater than 1.2, then the slope is likely the reason for the improved factor of safety even with the increased pore water pressures in the mitigated slope.

Secondly, the deep seated rapid drawdown critical circle for the F.S. = 1.2 appears unreasonable because the clay layer into which it penetrates is below the drawdown level, El 13. There should be no effect due to drawdown in this layer. The analysis should be checked to make sure an error has not occurred.

The TMA recommends the addition of a 35-foot deep cut off wall from the surface of a degraded existing levee for this segment. The text states on page 16 that the “bottom of the cutoff wall should extend at least 5 feet into the relatively firm clay as shown on Figure G-6—” and Exhibit G-6 (called a Figure in the quote) indicates the cutoff wall should extend to El -10. Review of the exhibit suggests that the bottom of the cutoff wall could be raised to the grades noted in the table below.

<table>
<thead>
<tr>
<th>Suggested Bottom of Cutoff Wall Grades</th>
</tr>
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<tbody>
<tr>
<td>Station</td>
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<td>0+00 to 15+00</td>
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<tr>
<td>15+00 to 26+00</td>
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<tr>
<td>26+00 to 43+00</td>
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</tbody>
</table>

BCI should consider the revised upward tip grades for the cutoff wall.

**Subsection 6.2.2 Segment B (Station 43+00 to 100+00) – Evaluation Cross-Section Station 79+00**

The TMA states on page 16 that an “adjacent levee alternative is proposed by the design team and WSAFCA for Segment B.” Specifically, BCI recommends an adjacent levee with a 250-foot-wide seepage berm to mitigate detrimental underseepage. We do not concur with this recommendation. The text also indicated that this recommendation did not apply to the first 1200 feet of the segment. BCI recommended the same solution as was used in Segment A for the first 400 feet of Segment B from Station 43+00 to 47+00. We concur with this recommendation. For the reach from Station 47+00 to 55+00, a deep cutoff wall to El -70 is recommended. This cutoff wall should be evaluated as an alternative but may not be necessary based on the following discussion.
As noted above in discussion of subsurface conditions for this segment, the surficial layer appears to be mischaracterized and appears to be much thicker that stated in the report. The portion of the segment under consideration is from Station 47+00 to 100+00. The TMA states on page 6 that “the upper 5 to 10 feet---consists predominantly of medium plasticity silt/clay with isolated zones of---high plasticity clay and---low plasticity sandy silt/sandy clay.” The layer is designated ML-CL on Exhibit G-XB, distributed at the BOSC meeting in February 2012. Below this layer is a low plastic silt and silty sand layer designed ML-SM on Exhibit G-XB indicating it is predominately fine grained silt. When these two layers are combined, the resulting layer extends to about El -0 and in some locations to El -20. It is also recommended that the ML-SM layer shown on Exhibit G-XB should extend deeper at the southern portion of the geologic profile from about Station 47+00 to 68+00. The fines contents of the various layers obtained from the boring logs along the center line of the segment are as follows:

<table>
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<th>Sample Elevation, ft</th>
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<th>15</th>
<th>10</th>
<th>5</th>
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<th>-5</th>
<th>-10</th>
<th>-15</th>
<th>-20</th>
<th>-25</th>
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The fines content values typically exceed 40 percent as shown by the **bold black** numbers in the table. The model used to evaluate this segment (Exhibit G25, Appendix F) was reviewed and the stratigraphy appears to be very conservative. The boundary between the upper ML-CL layer and the ML/SM layer has been adjusted downward based on review of the test boring and CPT sounding data as shown on the copy of Exhibit G25 below. Note Boring ENG-B-6 was added to provide additional clarity for the model.
Review of the seepage analyses (Exhibits G-26, G-28, G-29 and G-30) indicates that the permeability value used for the low plastic silt/silty sand (ML-SM) layer (6) was intermediate between that recommended by BCI for silty sand \((4 \times 10^{-4} \text{ cm/sec})\) and sandy silt \((2 \times 10^{-5} \text{ cm/sec})\) at \(2.1 \times 10^{-4} \text{ cm/sec}\). A more reasonable value of say \(8 \times 10^{-5} \text{ cm/sec}\) would seem more appropriate. The selection of this permeability value should be reviewed since it will have an impact on the exit gradient at the toe of the berm. The layer is also described on the above referenced Exhibits as silty sand to sand. This is incorrect as the ML-SM layer (6) contains predominately fine grained soils and the silty sands that are present typically contain more than 40% fines.

The end of construction stability cases (Exhibit G-34) include effective strength parameters for the low plastic silt/silty sand (ML-SM) layer (6) with high fines content: \(c' = 0 \text{ psf} \) and \(\phi' = 30^\circ\). This is incorrect. The recommended undrained shear strength for the low plastic silt is 500 psf.

Considering the rapid drawdown stability cases (Exhibits G-27 and G-33), it appears that the shear strength parameters for the low plastic silt/silty sand (ML-SM) layer (6) were selected assuming the layer is free draining. It is again mischaracterized as silty sand to sand on the referenced exhibits. This appears to be incorrect as the layer consists of both silts and silty sands with fines contents that typically exceed 40 percent. This material should not be considered free draining and both the effective strength and total strength parameters should be used in the analyses as follows: \(c' = 0 \text{ psf} \) and \(\phi' = 30^\circ\) and \(c = 240 \text{ psf} \) and \(\phi = 13^\circ\). This may impact the results of the analysis.

We understand that this segment has several constraints including: 1) the water side slope is steeper than the maximum 3H:1V, 2) the riprap is in poor condition, 3) vegetation exists on the water side slope, and 4) houses are very close to the landside toe of the levee. The BCI cross section at Station 79+00 indicates that the existing levee has a water side slope of 3H:1V, so this may not be a constraint. Given these constraints and the suggested revised stratigraphy, we propose that an alternate mitigation be considered for this segment.
The existing levee has a functioning landside drained stability berm and the exit gradient is \( i = 0.52 \) for the 200-year water surface elevation, El 32, shown on Exhibit G-26. The analysis was not performed for the HTOL water surface elevation, El 35, but it would be higher and the exit gradient would be unacceptable. It is recommended that the seepage analyses be redone for the revised stratigraphy shown above to see if the exit gradients are satisfactory.

If the exit gradient remains above the criteria, we recommend that a shallow cutoff wall to about El 0 to -5, as shown by the yellow line in the above table, be considered to further mitigate the excessive underseepage gradient. The slurry wall could be constructed by degrading the levee to the top of the existing drained stability berm and would effectively tie the upper layer together and form a blanket.

Relief wells should be considered in areas where it is not possible to construct the slurry wall due to clearance issues with homes. The blanket layer appears to be suitable for construction of a relief well system. If this suggested alternative proves satisfactory, it will save the expense of a new setback levee and the purchase of a significant number of costly homes.

Because of the suggestion to reconsider a mitigation in place approach for Segment B from Station 47+00 to Station 100+00, further review of the analyses were not completed.

We concur with the BCI assessment that significant long term settlement is not anticipated in this segment.

**Subsection 6.2.3 Segment C (Station 100+00 to 165+00) – Evaluation Cross-Section Station 122+00**

The TMA states on page 18 that “---a setback levee alternative is proposed by the design team and WSAFCA for Segment C.” Specifically, BCI recommends an adjacent levee with a 115-foot wide seepage berm to mitigate excessive underseepage. We concur with this adjacent levee mitigation recommendation. This BCI recommendation applies from Station 100+00 to 158+00. From Station 158+00 to the end of the segment at Station 165+00, BCI recommends use of the recommended mitigation for Segment D. As noted previously, the subsurface borings and CPT soundings suggest this boundary may be farther south to about Station 154+00.

Review of the subsurface conditions in the upper 10 feet of this segment (Station 100+00 to 158+00) consists of “--- silty sand and low plasticity sandy silt/sandy clay.” Below the surficial layer, the soils to a depth of 25 to 40 feet typically consisted of “---silty sand to---non-plastic sandy silt ---.” Review of the geologic profile of Exhibit G-XC suggests that the non-plastic sandy silt/silty sand (ML-SM) layer below the sandy silt/sandy clay (CL-ML) layer should extent to the north of the Boring BCI-11-B20 to Boring BCI-11-B3 at about El -10.

Review of the seepage analyses (Exhibits G-36, G-37, and G-38) indicates that the permeability value used for the silty sand/low plastic silt described as ML-SM layer (6) to be \( 2.1 \times 10^{-4} \) cm/sec. Based on the soil description approach discussed above under Segment A, it would seem more appropriate to designate this layer as SM-ML. The selected permeability value of \( 2.1 \times 10^{-4} \) cm/sec seems appropriate for this layer based on the lower fines content of the sands.

The exit gradients at the toe of the berm for the seepage model may be less than shown on Exhibits G-37 and G-38. The calculations indicate the hydraulic head at the ground surface is El 21.5. It appears that
the value should be greater than El 22 based on the data shown on the figures. BCI should check the output to verify the accuracy of the calculation.

We concur with the BCI assessment that significant long term settlement is not anticipated in this segment between Stations 100+00 and 158+00. The portion of the levee north of Station 158+00 will settle similar to the levee in Segment D and settlement of this section is discussion below.

Subsection 6.2.4 Segment D (Station 165+00 to 183+00) – Evaluation Cross-Section Station 167+00

The TMA states on page 20 that “---a setback levee alternative is proposed by the design team and WSAFCA for Segment D.” Specifically, BCI recommends a setback levee with a shallow, 25-foot deep, cutoff wall to mitigate potential underseepage. We concur with this setback levee mitigation recommendation. This BCI recommendation applies from Station 158+00 to 165+00 in Segment C and the full length of Segment D to Station 183+00.

The cutoff wall has minimal impact on the seepage gradient at the toe of the levee but it is reasonable to include it as it will provide an improved condition with respect to underseepage through unknown shallow sand layers typical of this geologic setting.

We also concur with the recommendation regarding settlement mitigation through the use of pressure relief drains (wick drains).

Subsection 6.2.5 Segment E (Station 183+00 to 214+00) – Evaluation Cross-Sections Station 197+35

The TMA states on page 20 that “---a setback levee alternative---is proposed by the design team and WSAFCA for Segment E.” Specifically, BCI recommends a setback levee with a shallow, 25-foot deep, cutoff wall to mitigate potential underseepage south of Station 197+00 and a 100-foot wide seepage berm north of Station 197+00. We concur with this setback levee mitigation recommendation.

The TMA indicates on page 8 that the subsurface conditions south of Station 197 in the upper 7 to 10 feet consist “---of silty sand and sandy silt.” This layer is underlain by 15 to 25 feet “---of medium to high plasticity clay.” North of Station 197+00, the upper 11 to 15 feet consists of “---low plasticity clay/silt and sand/silty sand---” which is underlain by “---silty sand and relatively clean sand---.” The difference in the two portions of this segment appears to be that the clay soils below the surficial soils are missing north of Station 197+00. Based on review of geologic profile of Exhibit G-XE, we do not concur with the change of subsurface conditions at Station 197+00. The addition of Boring BCI-11-B11 to the geologic profile (near BCI-11-B18 already included) suggests the transition should occur at about Station 193+00.

Review of Cross Section 197+35 on Figure G-52 suggests an alternative interpretation of the subsurface conditions as shown in the figure below. BCI should reconsider how the various strata are drawn on this figure. The truncation of the CL and CH layers in a vertical line is not reasonable.
Bringing the SP-SM layer closer to the ground surface as suggested in the figure may impact the results of the BCI seepage analyses shown on Exhibits G-53 through G-56. BCI should reevaluate its seepage analyses in light of these recommendations.

The cutoff wall has no apparent impact on the seepage gradients at the toe of the levee but it is reasonable to include it as it will provide an improved condition with respect to underseepage through unknown shallow sand layers typical of this geologic setting.

Review of the rapid drawdown and end of construction stability models on Exhibits G-59 and G-60, in light of the suggested modification to the geologic cross section on Exhibit G-52, suggests that the parameters for the ML layer should extend under the CL-CH layer.

We also concur with the recommendation regarding settlement mitigation through the use of pressure dissipation drains (wick drains) for the section of Segment E south of Station 197+00.

**Subsection 6.2.5 Segment E (Station 183+00 to 214+00) – Evaluation Cross-Sections Station 205+44**

The cross section at Station 205+44 shown on Exhibit G61 does not appear to relate to the subsurface investigation data. Additionally the symbols used on the stick log for Boring SE-B3 do not appear to indicate the same material as shown on the geologic profile of Exhibit G-10. It appears that the fine grained layer at the surface should not extend below about El 10. The SM layer should also not end in a vertical line. BCI should evaluate the need for these modifications. Reducing the thickness of the blanket layer may require a longer seepage berm.

Settlement of the portion of this segment north of Station 197+00 is considered under Segment F.
Subsection 6.2.6 Segment F (Station 214+00 to 273+00) – Evaluation Cross-Section Sta. 241+00

The TMA states on page 21 that “the design team and WSAFCA have selected an adjacent levee as the preferred alternative and a setback levee as the alternate alternative for Segment F---. BCI evaluated both alternatives---found---that a 100-foot-wide seepage berm---will mitigate underseepage.”

The TMA indicates on page 9 that the subsurface conditions in “---the upper 20 to 30 feet ---generally consists of ---low plasticity to non-plastic silt with lenses of silty sand---and with--- layers of low to medium plasticity clay and silt. Below 20 to 30 feet, the explorations indicate medium dense to dense sand with silt and gravel to about 90 feet depth.” Based on review of geologic profile of Exhibit G-XF, the upper shading and labeling of strata does not appear to be consistent with the stick logs. The upper CL-ML layer appears to be more extensive then the data indicates. The upper SM layer between Stations 206+00 and 218+00 (new stationing) appears to be likely an ML-SM layer. The same comments apply to the setback levee centerline data shown on Exhibit G-XG. This might better be labeled G-XFA as opposed to G-XG to avoid confusion with Segment G.

Review of Cross Section 241+00, shown on Exhibit G-69, suggests that Boring BCI-11-24, located on the crest of the existing levee, should be included for completeness. The section appears to represent the subsurface conditions based on review of the borings and CPT soundings.

The TMA states on page 21 that the exit gradient at the toe of the 100-foot berm is 0.9 for the water surface at the hydraulic top of levee (HTOL). The gradient shown on Exhibit G-74 indicates it is 0.74 for the adjacent levee. The gradient is 0.93 for the setback levee as shown on Exhibit G-81. The BCI text should be clarified.

Considering the rapid drawdown stability cases (Exhibits G-71, G-77 and G-84), it appears that the shear strength parameters for the low plastic silt/silty sand (ML-SM) layer (6) were selected assuming the layer is free draining. This appears to be incorrect as the layer consists mostly of both silts. This material should not be considered free draining and both the effective strength and total strength parameters should be used in the analyses as follows: c’ = 0 psf and φ’ = 30° and c = 240 psf and φ = 13°. This may impact the results of the analysis.

The end of construction stability cases (Exhibit G-78 and G-85) include effective strength parameters for the low plastic silt/silty sand (ML-SM) layer (6) with high fines content: c’ = 0 psf and φ’ = 30°. This is incorrect. The recommended undrained shear strength for the low plastic silt is 500 psf.

We concur with the BCI assessment that significant long term settlement is not anticipated in this segment.

Subsection 6.2.7 Segment G (Station 273+00 to 296+00) – Evaluation Cross-Section Sta. 283+00

The TMA states on page 22 that “the design team and WSAFCA have selected an adjacent levee alternative---” as the preferred alternative for Segment G. BCI’s further recommends “---a relatively deep 85- to 87-foot deep cutoff wall as shown on Figure G-12”---to--- “mitigate underseepage.”

The TMA indicates on page 9 that the subsurface conditions in “the upper 10 to 15 feet of soil---consists predominantly of low plasticity clay/silt with some zones of silty sand, which is underlain by silty sand to
depths of about 20 to 30 feet. Below this layer, relatively clean sand extends to depths of about 45 to 60 feet.” Review of the geologic profile of Exhibit G-XH (note the confusion discussed above about the labeling of this exhibit), distributed during the BOSC meeting in February 2012, indicates it appears to be a good representation of the subsurface conditions based on the boring and CPT sounding data.

Review of Cross Section 283+00, shown on Exhibit G-86, suggests that boundary between the SM-ML layer and the SP-SM layer should be lowered from about El 0 to about El -5. It also appears that a field boring should be drilled to better define the conditions on the landslide of the levee.

Considering the rapid drawdown stability cases (Exhibits G-88 and G-94), it appears that the shear strength parameters for the low plastic silt/silty sand (ML-SM) layer (6) were selected assuming the layer is free draining. This appears to be incorrect as the layer consists mostly of both silts. This material should not be considered free draining and both the effective strength and total strength parameters should be used in the analyses as follows: \( c' = 0 \text{ psf} \) and \( \varphi' = 30^\circ \) and \( c = 240 \text{ psf} \) and \( \varphi = 13^\circ \). This may impact the results of the analysis.

The end of construction stability cases (Exhibit G-95) include effective strength parameters for the low plastic silt/silty sand (ML-SM) layer (6) with high fines content: \( c' = 0 \text{ psf} \) and \( \varphi' = 30^\circ \). This is incorrect. The recommended undrained shear strength for the low plastic silt is 500 psf.

**Section 7.0 Continued Exploration and Analysis for TO4 Final Design**

Comments were provided under Section 6

**Section 8.0 Guidance Documents**

Comments were provided throughout previous sections.

**Section 9.0 Tables**

Comments were provided throughout previous sections.

**Section 10.0 Exhibits**

Note the title page for Appendix F indicates Cross Section 195+35 but should be 197+35

**Section 11.0 APPENDICIES**

Comments were provided throughout previous sections except for the following questions or comments.

1. Station 19+00. The gradient should have been calculated from bottom of the total blanket (clay layer on bottom) at about El -20.

2. Station 19+00 and all others. The steady state slope stability analysis should not be run toward the river. Why were these analyses performed?

3. Station 19+00. The levee saturation level for before drawdown is too high.
4. Station 19+00. Why is there such a large difference between the two sudden drawdown analyses? 0.91 and 1.2? (See discussion under Subsection 6.2.1)

5. Station 79+00. Why was the thickness of the ML/SM layer ignored? Should have transformed as per EM 1110-2-1913 and used this as total blanket.

6. Station 79+00. Why are different berm elevation used (22.89 and 23) in the computation on Exhibit G-29?

7. Station 79+00. The conservative high phreatic surface causes the low steady state FOS shown on Exhibit G-31. If k of the new levee is 0.0028 ft./day, it will take 4.89 years for the water to travel 5 feet into the levee. That is an unrealistically long flood.

8. Station 122+00. Why was the thickness of the ML/SM layer ignored? Should have transformed as per EM 1110-2-1913 and used this as total blanket.

9. Station 167+00. The gradient should have been calculated from bottom of the clay layer near El -10.

10. Station 195+00. Why was the thickness of the ML/SM layer ignored? Should have transformed as per EM 1110-2-1913 and used this as total blanket.

11. Station 205+00. This model should be closely reviewed. The SP-SM layer between approximately El 0 to -10 is acting like an aquifer but appears not to be directly connected. The real aquifer below El -90 should not pose a problem.

12. Station 241+00. Why was the thickness of the ML/SM layer ignored? Should have transformed as per EM 1110-2-1913 and used this as total blanket.

13. Station 241+00. The levee saturation level for before drawdown is too high. Also in the analyses with an adjacent levee, the sudden drawdown FOS was 1.0 and 1.5, where in the original levee it was 0.88. Why?

14. Station 283+00. The levee saturation level for before drawdown is too high.