DEPARTMENT OF THE ARMY
U.S. ARMY ENGINEER DIVISION, GREAT LAKES AND OHIO RIVER
CORPS OF ENGINEERS
550 MAIN STREET
CINCINNATI, OH 45202

CELRD-PD

29 Oct 13

MEMORANDUM for Huntington District, U.S. Army Corps of Engineers [redacted] 502 Eighth Street, Huntington, WV 25701-2070

SUBJECT: Major Subordinate Command (MSC) Approval of the Bolivar Major Rehabilitation Final USACE Response to Type II Independent External Peer Review (IEPR)

1. CELRH-EC, memorandum dated 7 August 2013, subject: Bolivar Major Rehabilitation – Final USACE Response to Type II Independent External Peer Review.

2. The Type II IEPR report and final written responses to the Safety Assurance Report have been reviewed by LRD Staff and found to be satisfactory. Approval to release the documents to the public is granted.

3. The point of contact for the MSC's approval is [redacted]

Encl

[redacted]

DEPARTMENT OF THE ARMY
HUNTINGTON DISTRICT, CORPS OF ENGINEERS
502 EIGHTH STREET
HUNTINGTON, WV 25701

REPLY TO
ATTENTION OF

CELRH-EC

7 August 2013

MEMORANDUM FOR Great Lakes & Ohio River Division, CELRD-RBT [redacted] 550 Main Street, Room 10032, Cincinnati, OH 45202-3222

SUBJECT: Bolivar Major Rehabilitation – Final USACE Response to Type II Independent External Peer Review

1. A Type II Independent External Peer Review (IEPR) Safety Assurance Review (SAR) was conducted for the subject project in accordance with Section 2035 of the Water Resources Development Act of 2007, EC 1165-2-214, and the Office of Management and Budget's Final Information Quality Bulletin for Peer Review (2004).

2. The IEPR was conducted by Battelle Memorial Institute. The IEPR Panel consisted of six panel members with technical expertise in geotechnical engineering, instrumentation engineering, hydraulic engineering, engineering geology, civil engineering, and economics.

3. The enclosed document contains written responses to the IEPR comments from five separate reviews conducted during the design process for the Bolivar Major Rehabilitation Project:

Major Rehabilitation Report (MRR)
Design Documentation Report (DDR)
90% Seepage Barrier Plans and Specifications (P&S)
100% Service Gates Replacement P&S
100% Seepage Barrier P&S.

4. The SAR report and final written responses to the SAR are submitted for review and concurrence. The IEPR Report and the USACE responses will be made available to the public on the Huntington District's website following approval by BG Burcham.

5. If there are any questions or comments on this matter please contact [redacted]

[redacted]

encls

Bolivar Dam, Ohio'
Major Rehabilitation Report,'
Major Rehabilitation Design Documentation Report,'
Service Gates Replacement Plans and Specifications,'
and'
Seepage Barrier Plans and Specifications'

FINAL'
U.S. Army Corps of Engineers Response to'
Type II Independent External Peer Review (Safety Assurance Review)'
August 2013

A Type II Independent External Peer Review (IEPR) Safety Assurance Review (SAR) was conducted for the subject project in accordance with Section 2035 of the Water Resources Development Act of 2007, EC 1165-2-214 Water Resources Policies and Authorities – Civil Works Review, and the Office of Management and Budget's *Final Information Quality Bulletin for Peer Review (2004).*

The goal of the U.S. Army Corps of Engineers (USACE) Civil Works program is to always provide the most scientifically sound, sustainable water resource solutions for the nation. The USACE review processes are essential to ensuring project safety and quality of the products USACE provides to the American people. The Type II IEPR is conducted on design and construction activities for any project where potential hazards pose a significant threat to human life (public safety). This applies to new projects and to the major repair, rehabilitation, replacement, or modification of existing facilities.

Battelle Memorial Institute (Battelle), a non-profit science and technology organization with experience in establishing and administering peer review panels for USACE, was engaged to conduct the Type II IEPR of the Major Rehabilitation Report (MRR), Design Documentation Report (DDR), Service Gates Replacement 100% Plans and Specifications (P&S), the Seepage Barrier 90% P S, and the revised left abutment design as reflected in the Seepage Barrier 100% P&S. The only portions of the MRR reviewed for this IEPR covered the economic assessment of the alternatives per Type I IEPR Waiver dated 10 Apr 2009 – see Attachment No. 1.

Table 1 summarizes the comments received during review of the MRR, DDR and P&S products. 129 total comments were identified and documented, of which 57 were initially identified as "critical" comments. Critical was defined as any component, subcomponent, or system whose malfunction can cause a cascading failure of the entire structure and pose a risk of serious injury, loss of life, or loss of mission objectives.

TABLE 1. Summary of Comments by Review'

REPORT	TOTAL COMMENTS	CRITICAL COMMENTS
Major Rehabilitation Report	12	4
Design Documentation Report	57	37
100% Service Gates Replacement P&S	7	2
90% Seepage Barrier P&S	31	13
100% Seepage Barrier P&S	22	1
TOTAL	129	57

The IEPR reviews began in June 2010 with the MRR and DDR following in descending order the products listed in Table 1. The USACE Project Delivery Team (PDT) responses sufficiently addressed all Battelle IEPR Panel concerns, the comments were closed out and a Final IEPR Design Review Report was issued on 21 Dec 2012. See accompanying report titled **Final Independent External Peer Review Design Review Report for the Assessment, Analysis, and Evaluation of the Bolivar Dam Safety Assurance Project.**

All comments and responses were documented in the Design Review and Checking System (DrChecks). Reports generated from this system are shown in the appendices which follow with IEPR Panel comments formatted in bold font followed by the PDT evaluation of the comment. The PDT evaluations include explanations for "Non-concurrence" or a description of the action to be taken for "Concurrence". In some cases a reviewer simply asks a question in which a response is given as "For Information Only". Of the 129 comments addressed during the IEPR reviews, the PDT concurred with 106, non-concurred with 14, and responded to 9 with information only.

LIST OF APPENDICES

ATTACHMENT No. 1: Type I IEPR Waiver'

DEPARTMENT OF THE ARMY
U.S. ARMY ENGINEER DIVISION, GREAT LAKES AND OHIO RIVER
CORPS OF ENGINEERS
550 MAIN STREET
CINCINNATI, OH 45202

APR 10 2009

CELRD-DE

MEMORANDUM FOR Commander, Huntington District (CELRH-DE), 502 Eighth Street, Huntington, WV 25701-2070

SUBJECT: Request for Waiver of Type I Independent External Peer Review (IEPR) of Bolivar Dam Major Rehabilitation Report

1. Reference:

   a. CELRH-DE memorandum, same subject, dated 26 January 2009

   b. EC 1105-2-410, 22 August 2008, Review of Decision Documents

   c. EC 1165-2-209 Draft, 6 January 2009, Civil Works Review Policy

2. Reference 1.a. provided a summary of Bolivar Dam Major Rehabilitation Report and requested a waiver of a Type I Independent External Peer Review (IEPR) for that report.

3. Paragraph 6.d. of references 1.b. states that the decision to conduct an IEPR rests with the MSC Commander. Furthermore, paragraph 7.f. of reference 1.c. says, "the vertical team (involving district, MSC, RMO, and HQ members) will advise the MSC Commander as to whether IEPR is appropriate. The decision to conduct IEPR rests with the MSC Commander."

4. Based on advice from the vertical team, I have determined that a Type I IEPR is not required for Bolivar Dam Major Rehabilitation provided that:

   a. The work does not require an Environmental Impact Statement (EIS);

   b. The work is within the footprint of the existing dam;

   c. The work is for an activity for which there is ample experience within the Corps of Engineers and industry to treat the activity as being routine; and

CELRD-DE
SUBJECT: Request for Waiver of Type I Independent External Peer Review (IEPR) of Bolivar Dam Major Rehabilitation Report

d. A Type II, Independent External Peer Review (Safety Assurance Review), with the addition of a review of economics of the alternatives, is started as one of the first activities in the design phase of the modification.

5. [redacted]

[redacted]

APPENDIX A

MAJOR REHABILITATION REPORT IEPR

Comment ID	Discipline	Section/Figure	Page Number	Line Number
IEPR Comment 3421687	Economics	Main Report, Section 5	n/a	n/a

The without project condition does not appear to address the replacement of the closure system described in Section 3 (pages 11-12), nor does it address the interim operation plan described in Section 4. The future without project condition should include all reasonably foreseeable actions to avoid or mitigate damages caused by the water resource problem being analyzed. Rehabilitation of the gate system would appear to be one such reasonable action. If the gate structure fails to close, then no projects undertaken to control seepage will result in retention of flood flows. If the gate fails to open, then the dam's integrity may be compromised and the stream flows uncontrolled. The interim operation plan should also be a part of the without project condition, since Section 4.0 describes it as the most likely operational regime. The report cannot identify a set of expected future conditions (i.e, future gate repairs and interim operation plans) and analyze a different set of conditions as the future without project. Significant changes in the economic losses described in the future without project condition could have significant effects on the recommended plan's economic feasibility. To resolve these concerns, the report would need to include: 1. A full accounting of the expected future costs associated with gate component replacement. 2. A full accounting of flood damage, recreation and other costs associated with the interim operating plan.

1-0 Evaluation **Non-concurred**

The replacement of the gates and operating machinery (closure system) was not addressed in the without project condition due to its relatively minor scope, when compared with the geotechnical aspects of the project. According to EP 1110-2-13, Dam Safety Preparedness, "The Major Rehabilitation Program is limited to the major repair or restoration of main structures such as dams, locks, and powerhouses, exclusive of electrical, mechanical, and other equipment, except that such equipment may be included where it is essential to and integral with the feature of the project being rehabilitated." Given this guidance, rehabilitation of the "closure system" would have been added to any recommended alternative. When compared to the cost for the rehabilitation of the spillway and main embankment, the cost for the "closure system" is negligible, and is not likely to impact the economic justification whether included or not. Likewise, when compared to the probabilities of failure for the spillway and main embankment, the probabilities of failure for the "closure system" are also negligible and would not have had a defining impact on the recommended alternative. As for the inclusion of the Interim Operating Pool (IOP) with the base condition, the formulation of this project was consistent with guidance and previously approved Rehabilitation Evaluation Reports. In most MRR studies across the division at the time of project formulation, IOPs were not considered as part of the base condition, or without project condition, for several reasons. The IOP and its associated Interim Maximum Flood Control Pool (IMFCP) are meant only as temporary non-structural measures to reduce the risk of dam failure until a structural fix is implemented. This IMFCP is not considered a permanent fix now and was not considered a permanent fix in the feasibility document. Moreover, this measure, along with the entire IRRM Plan, was not approved until 18 months after the completion of the Bolivar MRR. The District included a discussion of IRRMs within this document as a means to inform the public about our continued efforts to decrease the risk of failure at the dam. The guidance for the Rehabilitation Evaluation Report (EP1130-2-500) does not direct the District to assess non-structural alternatives. If the project had not been approved, the District may have proceeded with a feasibility study on permanently lowering the flood control pool. To permanently adopt the IOP would require the District to undertake a separate Sec 216 "Review of Completed Projects" study.

1-1 Backcheck Recommendation **Close Comment**
The panel recommends that the report be modified and the discussion of the without project condition include a condensed version of the USACE evaluation to demonstrate compliance with ER 1105-2-100 guidance.

IEPR Comment 3421696	Economics	Main Report, Section 8.5	n/a	n/a

This section of the report identifies four initial alternatives but the subsections following only describe three, apparently omitting a description of scheduled repair. The future without project condition should include all reasonably foreseeable actions to avoid or mitigate damages caused by the water resource problem being analyzed. The omission affects only the technical completeness of the report and does not appear to be likely to affect the selection of the recommended plan. To resolve these concerns, the report would need to include: 1. Include a discussion of a scheduled repair strategy and explain why it would or would not be carried forward for additional analysis.

1-0 Evaluation **Concurred**
A discussion of the scheduled repair strategy and the rationale for its screening from potential alternatives will be added to the Main Report.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421699	Economics	Main Report, Section 8.5.5.1	n/a	n/a

Rehabilitation costs described elsewhere in the report are inconsistent with the $70 million figure here. The figures expressed in the report should be internally consistent with one another, as well as maintain consistency with the data, tables and figures shown in the various Appendices Such inconsistencies affect only the technical quality of the report and

are not likely to have an effect on the selection of the recommended plan. To resolve these concerns, the report would need to include: 1. Review the report and Appendices and modify as needed to maintain internal consistency.

1-0 Evaluation **Concurred**

The report and technical appendices will be reviewed, the accuracy of the rehabilitation costs checked, and corrections made where necessary.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421704	Economics	Main Report, Section 8.5.5.2	n/a	n/a

Consistent with the concern expressed in Comment 3421687, this discussion does not convince the reader that the future without project condition will not include a limited operational capacity. The basis for comment is ER 1105-2-100: Only one base condition--the future without project condition--can be used to evaluate the effectiveness of alternative plans. Section 4.0 of the main report states that an interim operating condition would require floodwaters to be released earlier than the current operating plan to prevent pool levels from exceeding unsafe elevations that would threaten the integrity of the dam. This is likely to increase future without project condition damages and effect the justification of the recommended plan. The basis for not including the interim operations plan as part of the without project condition appears to be that it will not affect dam stability during high flow events. While this type of all-or-nothing approach simplifies the task of computing without project condition damages, the resultant without project condition does not accurately reflect the expected real-world future. The report cannot identify one expected future conditions (i.e, interim operation plans) and analyze a different set of conditions as the future without project. Significant changes in the economic losses described in the future without project condition could have significant effects on the recommended plan s economic feasibility. To resolve these concerns, the report would need to include: 1. Include a full accounting of flood damage, recreation and other costs associated with the interim operating plan.

1-0 Evaluation **Non-concurred**

The IOP and its associated Interim Maximum Flood Control Pool (IMFCP) are meant only as temporary non-structural measures to reduce the risk of dam failure until a structural fix is implemented. As for the inclusion of the Interim Operating Pool (IOP) with the base condition, the formulation of this project was consistent with Major Rehabilitation guidance (EP1130-2-500) and previously approved Rehabilitation Evaluation Reports. As such, these IRRMs were not considered permanent fixes during the development of this feasibility document and removed from consideration in the without project condition. As stated in the response to Comment #3421687, if the project had not been approved, or if rehabilitation were determined not to be feasible, the District may have proceeded with a feasibility study on permanently lowering the flood control pool. Congressional authorization would be needed to establish a permanent reduction in operational capacity, and would be studied under the Section 216 authority. Moreover, though discussed in the report, the

IRRMs were only preliminary proposed measures at the time of report preparation. The IRRMs were finalized and approved at Division Headquarters in February of 2010.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421712	Economics	Main Report, Table 6	n/a	n/a

This table appears to have inaccurate calculations for net benefits. The basis for this comment is technical completeness and accuracy. Subtracting average annual costs from average annual benefits does not produce the values shown for net benefits.Such inconsistencies affect only the technical quality of the report and are not likely to have an effect on the selection of the recommended plan. To resolve these concerns, the report would need to include: 1. Revise the report to reflect mathematically accurate calculations.

1-0 Evaluation **Concurred**

The report will be revised to show accurate calculations in all tables.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421715	Economics	Main Report, Tables 6-9 and unnamed table, page 60	n/a	n/a

These tables' header indicate that the figures shown are in FY06 dollars or FY08 dollars rather than FY10, or have no year shown at all. The basis for comment is 1105-2-100. All values should be shown in dollars representing no less than three fiscal years of the date of the report. While the date of the draft report is FY09, the date of the final will be at least FY10. Table 6's figures will then be obsolete, requiring a reevaluation of project benefits and project costs. Such inconsistencies affect the completeness and understanding, but could rise to a higher significance if not timely corrected. To resolve these concerns, the report would need to include: 1. Update the values to current fiscal year dollars using appropriate indices or data series.

1-0 Evaluation **Concurred**

As stated in ER 1105-2-100, it is Corps policy to report and maintain current estimates of project benefits, costs and economic justification of all active funding projects and separable elements. However, the Bolivar Major Rehabilitation Report has been approved by the Great Lakes and Ohio River Division, and the project has moved into the DDR (Detailed Design) phase. At the time of the Rehabilitation Report's approval the figures in the report were in FY08 dollars, and considered current by the standards set forth in the ER. Also, in accordance with the ER, the Huntington District maintains, internally for yearly budgeting purposes, updated project costs and benefits for each of its projects. These are not incorporated within the approved report. Only if the Bolivar Major Rehabilitation Report had not yet been approved, would be appropriate to revise the numbers in the draft of the document. The Corps does not typically revise an approved decision document to show updated economic figures every three fiscal years. The report will be revised to show all numbers in FY08 dollars, and remove any inconsistencies between FY06 and FY08 dollars.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421720	Economics	Main Report, Section 8	n/a	n/a

The report does not appear to give consideration to the potential decrease in O&M costs as suggested in Section 9.9 The basis for comment is ER 1105-2-100: Only one base condition--the future without project condition--can be used to evaluate the effectiveness of alternative plans. Section 4.0 of the main report states that an interim operating condition would require floodwaters to be released earlier than the current operating plan to prevent pool levels from exceeding unsafe elevations that would threaten the integrity of the dam. This is likely to increase future without project condition damages and effect the justification of the recommended plan. The basis for not including the interim operations plan as part of the without project condition appears to be that it will not affect dam stability during high flow events. While this type of all-or-nothing approach simplifies the task of computing without project condition damages, the resultant without project condition does not accurately reflect the expected real-world future. Affects the completeness or understanding of the report, and could result in the report demonstrating that the project has even greater value to the nation. To resolve these concerns, the report would need to include: 1. Estimate without project condition O&M costs. 2. Estimate O&M costs under the recommended plan. 3. Compute net O&M costs and display the difference in Section 8.

1-0 Evaluation **Non-concurred**

Please see Comments #3421704 and #3421687 for a discussion on why the IOP was not included in the without project condition. Appendix B, Section 6.2 states "There are no additional O&M costs associated with the preferred project alternative." Therefore any change in O&M costs under the with project condition would be a decrease in O M costs. These costs (benefits), when compared with the other benefits of the project are negligible, and their exclusion only serves to under estimate the net benefits of the project, and were not developed for the with project condition for this reason. As stated in response to Comment #3421724, if any O M costs for the recommended alternative are developed, they will be included in future economic updates.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421724	Economics	Appendix B, Section 2	n/a	n/a

The concerns expressed in Comments 3421687 and 3421704 are reiterated here, in that the

future without project condition should include the interim operations plan. The basis for comment is 1105-2-100. All reasonably significant NED benefit categories should be documented, analyzed and displayed. If O&M costs are expected to be significantly lower under the recommended plan, the reduction in these costs is a legitimate NED benefit and should be included. Significant changes in the economic losses described in the future without project condition could have significant effects on the recommended plan's economic feasibility. To resolve these concerns, the report would need to include: 1. Include a full accounting of flood damage, recreation and other costs associated with the interim operating plan.

1-0 Evaluation **Non-concurred**

Please see response to Comments #3421704 and #3421687 for a discussion on why the IOP was not included in the without project condition. Please see response to Comment #3421720 for a discussion on why O M costs were excluded from the economic analysis.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421729	Economics	Appendix B, Tables 17 and 21	n/a	n/a

The fact that the figures in the tables do not sum horizontally is somewhat confusing. The basis for comment is technical clarity. Intuition leads the reader to expect the values for the separable components to sum horizontally. A relationship between the economic performance of the components being analyzed is not presented or discussed in the text, leading the reader to wonder why these figures don't add up. Affects only the clarity and understanding of the report. To resolve these concerns, the report would need to include: 1. Additional text in the report to describe the relationship between the abutment and main embankment leading to the values not summing horizontally.

1-0 Evaluation **Concurred**

The tables do not sum horizontally because the vertical columns represent the rehabilitation cost including mobilization and demobilization. When the column header reads, for example, "main embankment only," that includes the cost of rehabilitating the main embankment alone, and includes the cost of mobilization and demobilization. When the column header reads "both" that includes the cost of rehabilitation both components of the dam (main embankment and left abutment), as well as the cost of mobilization and demobilization. There is a cost savings associated with rehabilitating both components of the dam at the same time, because the contractor would only have to mobilize and demobilize one time, rather than two separate times, should the rehabilitation of the components take place at different times. Text will be added to the document to clarify this.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421744	Economics	Appendix B, Section 7.2.2	n/a	n/a

More information is needed on how the relationship between Bolivar and Dover Dam is treated in the analysis. The basis for clarity is technical clarity. This section appears to state that for the seven-year period until the Dover Dam rehab is complete, the relationship is such that base condition damages are lower than if Dover Dam were unaffected by a failure of Bolivar. Elsewhere, the report states that a failure at Bolivar would likely result in a corresponding increase in the risk of failure at Dover. If so, then for the seven year period of Dover's rehab construction, the potential damages reduced by this rehab project would be higher, not lower. Affects technical accuracy and clarity and is not likely to affect the selection of the recommended plan. To resolve these concerns, the report would need to include: 1. Clarify this section and/or the sections of the main report discussing the relationship between the two structures, such that the reader understands that as long as both dams face risk of failure, potential future damages are higher than if Dover is complete.

1-0 Evaluation **Concurred**

The base condition damages for Bolivar are higher during the seven year period until the Dover Dam rehab is complete. For the purpose of our analysis we assumed that for the first seven years of evaluation, if Bolivar failed, then Dover would fail in turn. The report will be revised for clarity on this point, and any conflicting sections will be corrected.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421751	Economics	Appendix B and Addenda 1 2	n/a	n/a

More information is needed on how the return frequency of the geotechnically significant pool levels were developed and incorporated into the analysis. The basis for comment is 1105-2-100, EM 1110-2-1619. The appendix and addenda identify a suite of pool levels determined to have significant concern from a geotechnical standpoint, and the consequences of encountering those pool level events are properly described and well documented. However, little information is provided on the frequency associated with these pool level events. Accordingly, it is difficult to determine how the frequency relationships are developed in the analysis. This currently affects the completeness and understanding of the report, but could rise in significance if the relationship between pool levels and frequency has not been thoroughly developed. To resolve these concerns, the report would need to include: 1. Clearly describe the hydrologic assumptions regarding frequency of flood events associated with a given pool level. 2. Provide clear, understandable linkage between flood frequency and expected annual damage.

1-0 Evaluation **Concurred**

Information about how the return frequency of the geotechnically significant pool levels can be found in Appendix H to the Main Report. Once the array of pool elevations were selected, Huntington District H&H team members provided water surface profiles for the with and without project condition for each of the pool elevations. These water surface profiles were imported into HEC-FDA which was used to calculate the per-event damages for each of these pool elevations for the with and without project condition. The per-event damages calculated by the HEC-FDA model were then used as input to the risk and uncertainty model developed by Pittsburgh District, which in turn provided expected average annual damages which were linked to the reliability of the dam. See response to comment # 3421764 for further details.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3421764	Economics	Appendix B, Addenda 1	2	n/a	n/a

More information is needed on how HEC-FDA was used in this analysis and how the damage-frequency relationships were developed. The basis for comment is 1105-2-100, EM 1110-2-1619. The appendix and accompanying addenda do not clearly describe how HEC-FDA was incorporated into the analysis. The addenda appear to show that HEC-FDA was used only to develop a set of stage-damage functions for later use in the decision tree model. If this is the case, then the base condition and with rehab condition damages may not represent the integration of a damage-frequency relationship, and are thus not representative of expected annual damages, as required by existing USACE guidance. This could be a fundamental problem with the analysis if it does not present expected annual damages, and it could affect the selection or justification of the project. To resolve these concerns, the report would need to include: 3. Clearly explain the level to which HEC-FDA was used in this analysis, and whether HEC-FDA was used to calculate expected annual damages. 4. Clearly explain how values derived from HEC-FDA were used in the decision-tree analysis, and how those values serve as a reasonable substitute for HEC-FDA produced values.

1-0 Evaluation **Concurred**

The stage damage relationships were developed using the HEC-FDA model which is the classic model used by the Corps for this purpose. The reviewer is correct that the damages for different frequency events were not integrated to compute expected average annual damages in the traditional manner described in EM 1110-2-1619 because the analysis is not the traditional new project study that is the basis for the procedure. The procedure for rehabilitation studies is described in ER 1130-2-500 and requires a life cycle analysis that considers the occurrence of different pool elevations and, for each pool elevation, the probability of unsatisfactory performance, including failure, and the possible flood damages. Particular events are simulated by generating random numbers and comparing them to probabilities for the occurrence of specific events as developed by the engineering staff. Upon completion of twenty thousand iterations of the life cycle simulation, the flood damages were averaged, discounted, and annualized. In sum, possible flood damages were simulated over a 50 year life cycle rather than considered as constants as assumed in the traditional analysis. As part of the validation of the procedure employed in this study, the expected flood damages using the traditional method were compared to those computed from the output of the simulation model for the first year in the analysis and the values were within 5 percent. This indicates that the procedure used in the study is not only appropriate, but consistent with the classic method. Appendix B will be revised to for technical clarification.

1-1 Backcheck Recommendation **Close Comment**

APPENDIX B

DESIGN DOCUMENTATION REPORT IEPR

Comment ID	Discipline	Section/Figure	Page Number	Line Number
IEPR Comment 3437827	Geotechnical	2.3.2	9	n/a

The DDR-for ATR indicates on page 9 that water enters coal/underclay/limestone at approximately elev 935 on the upstream face of the abutment and that uncontrolled seepage has been observed with pool elevations in excess of elev. 940. This document indicates "The extent of the seepage path through the abutment bedrock cannot be verified; therefore, the future integrity of the abutment/embankment contact is in question." It has been indicated the plan is to radial grout the left abutment and install a concrete seepage barrier, consisting of hydromill panels, to a depth of three feet into competent bedrock. Although significant borehole, dye tests, coring, etc. have been performed how will the USACE confirm the hydromill panels have been installed sufficient distance into "competent bedrock"?

1-0 Evaluation **Concurred**

The reviewer is referred to the response for comment 3437833 which should be a suitable response for this comment as well.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437833	Geotechnical	6.1.5	26	n/a

This section of the DDR indicates "Across this 500 foot transition area, the base elevation of the barrier will be stair-stepped upwards from 815 feet to 900 feet, with a minimum embedment of 3 feet into competent rock at the left abutment achieved." What procedures will be used in construction to visually confirm that competent rock has been reached?

1-0 Evaluation **Concurred**

As discussed during conference call with reviewers, the elevation of unweathered/competent rock has been defined to some extent to date by a number of borings at the left abutment. The resolution of this definition will be significantly increased prior to barrier installation by performing an adequate number of additional borings across the exact barrier alignment where stair-steping will occur. During construction, excavation equipment and instrumentation will provide accurate and real-time information concerning depth/elevation of excavation/embedment acheived, and through evaluation of these data, excavation resistance by equipment, real-time visual inspection of material being excavated, post-barrier installation coring, etc., adequate embedment into competent rock will be achieved and verified.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437840	Geotechnical	Geotechnical Analysis App B	18	n/a

This section indicates "To complete probabilistic slope stability analyses of the embankment and natural terrace for Bolivar Dam's profiles and load cases, SLOPE/W was utilized. The program generated both deterministic (as described above), and then probabilistic results for slope failures utilizing Monte Carlo simulation (GEOSLOPE International Ltd., 2004), which is a module built into the software. The unit weight of the embankment, terrace, and foundation soils, along with ??values of these materials, were treated as random variables with normal distributions within the range between parameters' HCV and LCV. The expected and probabilistic parameters in Figure 5 were used for obtaining deterministic FS(progressive erosion) and Pr(u) values. The Monte Carlo simulation runs thousands of slope stability "trial" analyses (5000 iterations were determined appropriate and run for each model in this work) on the selected failure surface while randomly varying the input soil properties within the specified ranges."I did not see any indication in any of the documents where either a hand-check or second software program were used to independently check the results of the Geoslope computer runs for final critical surfaces. While Geoslope Slope/W software is a good program I have found instances where the software incorrectly calculates factor of safety as discovered during use of hand-checks of critical surfaces. Have you performed independent checks of the critical surfaces either with a second software program or other means?

1-0 Evaluation **Concurred**

Hand calculation checks on infinite slope failures generated by Slope/W have previously been performed for shallow failures with horizontally emerging seepage, and these hand checks agreed well with Slope/W results. These independent checks were not included in the DDR but they will be included in the revised version. Additionally, hand calculation checks will be performed for representative deeper-seated critical failures to ensure agreement between independent approach and Slope/W results is reasonable; these will also be included in the revised DDR.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437848	Geotechnical	Geotechnical Analysis App B	n/a	n/a

The slope stability analysis presented in this appendix analyzes several conditions. Of particular concern is that it has been noted the designer relies on tailwater being present downstream of the Bolivar dam and it has been indicated that lower factor of safety would be calculated without this. Have you analyzed the risk of progressive dam failures? What happens if Dover dam fails and you cannot rely on the tailwater to improve stability of Bolivar dam? I did not see any discussion of this risk in the analysis.

1-0 Evaluation **Concurred**

Subsequent to the draft DDR preparation, transient seepage and slope stability modeling has been performed utilizing expected pool and tail water elevation hydrographs for extreme loading event at Bolivar Dam. This work has focussed on gaining broader understanding of potential slope instability during extreme loading event by considering stability at different time steps during the event (during which tail water elevations vary across a range). This work will be included in the revised DDR, and to an extent addresses the review comment. The risk of progressive dam failures as suggested has not explicitly been analyzed/quantified as part of this DDR. As part of current risk assessment processes (ongoing potential failure mode analyses and risk assessment studies), we do not currently consider progressive dam failures. While possible, the probability of slope instability at Bolivar Dam due to failure of Dover Dam is at worst (if one were to assume slope failure of Bolivar Dam could be certain upon Dover failure) equivalent to the probability of failure of Dover Dam. The expected long-term condition for this hypothetical analysis is that Dover Dam will exist in its remediated state such that it is in accordance with tolerable risk guidelines (i.e. have tolerable annual probability of failure). Therefore, it seems that while this is a credible failure mode, it is an insignificant risk driver, and it does not pose an intolerable annual probability of failure.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437860	Civil	Appendix B	Para. 2.2	n/a

Right abutment has apparently had little analysis other than flow net calculations at sta. 5+00. Although the embankment here is only 20' high, it has never experienced a hydraulic head so we don't know how it will react when in service. The gravel layer rises steeply as it approaches the right abutment and potentially could be a seepage path around/beneath the embankment. Has the potential of an abutment failure been considered or analyzed, and if not, I suggest giving a potential right abutment failure mode some thought.

1-0 Evaluation **Concurred**

Subsequent to the IEPR, we have been working in conjunction with an external USACE risk cadre, and through this work we are performing potential failure mode and risk assessment analyses. The right abutment is receiving consideration/analysis as suggested by this review comment. This work is ongoing, and the DDR will be revised as necessary depending on the outcome of these efforts.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437867	Civil	BG 708	n/a	n/a

Embedment of the cut off wall is shown to be embedded 3 feet into competent rock. Suggest that "competent rock" be defined, or specify a field inspection and approval by USACE personnel to determine if the excavation is into competent rock. This information should also be shown on drawing CG302, or construction drawings when they are developed.

1-0 Evaluation **Concurred**
The reviewer is directed to the response for comment 3437833, which was a similar comment. The response for that comment should satisfy this comment as well.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437875	Civil	DDR	21	4-5

Bank run sand is specified here and on dwg CG501. From narrative in the DDR, the borrow pit sand and gravel is expected to satisfy design requirements for a free draining material. Suggest that construction quality control procedures result in confirmation that the borrow pit material meets the intended gradation, and develop construction QA/QC measures to insure it is free draining.

1-0 Evaluation **Concurred**
Construction QC/QA procedures will confirm that the borrow pit material meets intended gradation as review comment recommends.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437881	Civil	Main Report	18	n/a

I did not see mention of environmental protection controls, including construction site erosion control, storm water management during construction, or requirements for control of sediments or runoff from the bentonite slurry construction site. I assume that these will be addressed in the plans and specifications phase. Please confirm.

1-0 Evaluation **Concurred**
Added the paragraph shown below to the Main Report. "7.2.5 Environmental Protection The contractor will be required to manage storm water, prevent erosion, and control sediments from the site during construction through the use of erosion and sediment control measures, and obtaining the Ohio EPA General Storm Water Permit for Construction Activity. Silt fencing, ditch checks, and temporary seeding have been incorporated into the quantities for the DDR. Details of the environmental protection features will be developed in the during the plans and specifications phase of the project."

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437888	Civil	CS101/Exh. C-2	n/a	n/a

Contractor staging/laydown area is designated for the seepage barrier construction, however, no construction staging area is designated on the downstream side of the dam.

Because of the extensive drainage and seepage features downstream from the dam, I suggest that a staging area be designated so that you can control traffic and construction activities to avoid damage to the drainage features.

1-0 Evaluation **Concurred**
A second staging area has been designated on the downstream terrace area as shown on Exhibit C-2.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437895	Civil	CG501/Exh. C-10 C-11	n/a	n/a

The title of these drawings indicates that there would be seepage blanket details found on the drawings, but the drawings appear to address the seepage barrier only. Cross sectional drawings of the seepage barrier and terrace filter blanket would clarify what features are existing and what features will be new as well as how they tie together.

1-0 Evaluation **Concurred**
Will remove "And Seepage Blanket" from the title block of Exhibits C-10 and C-11. A new section showing the terrace filter blanket extension will be added to the drawings.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437904	Civil	CG501/Exh C-12	18	n/a

There is a note in section A-3 on the drawing stating there are to be relief trenches excavated through the alluvial blanket and extending 1' into pervious material to relieve seepage pressure. I suggest that you define "pervious material," or specify that an USACE inspector review and approve excavation depth into pervious material so that we get the intended results.

1-0 Evaluation **Concurred**
Agree with the suggestion. Pervious material will be further defined and an inspector will in fact review/approve excavation depths during construction.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437917	Civil	n/a'	n/a	n/a

The successful operation of the flood control project depends on the ability to operate gates during a flood event. When the gates are out of service for rehabilitation, you will be limited in your ability to control flows. Large flows are rare, but I suggest you develop revised gate operational procedures to be used during construction when one or more gates are out of service.

1-0 Evaluation **Concurred**

Design Branch has initiated the coordination with Operation Division and considered the potential impact of the gate replacement to the project operation. The project plans and specifications include the procurement of a second bulkhead, necessary to address emergencies, contingencies and facilitate construction. Contract documents will detail restrictions, construction limits, and require full coordination with Operations Division during construction. Shop drawing submissions will include construction schedule and sequence.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3437984	Geotechnical	Design Documentation Report (DDR)	1&	n/a

The report states "Due to history of excessive seepage through (underlining is mine) and under the dam and through the left abutment during events with frequent return periods, it was ranked by U.S. Army Corps of Engineers ----". In Appendix B on Geotechnical Analysis – Embankment, Section 2.2 "Embankments and Abutments", it is stated "No through seepage-related concerns have been observed to date for the dam s rolled earth embankment". Comment: The above two statements are somewhat contradictory and confusing. From the first statement alone it is not evident that the seepage occurs only through the foundation material. Also, throughout the report, the terms "through seepage" and "under seepage" are used. In Appendix B on "Geotechnical Analysis – Embankment", these terms are used in the first section on "Overview" (Section 1.0). However, the difference between these two types of seepage is not defined anywhere in the report (unless I missed it). Since the rehabilitation plan is all about preventing these two types of seepage, a clearly defined distinction between these two types of seepage will be helpful.

1-0 Evaluation **Concurred**

In hindsight the use of the term through-seepage to describe horizontal flow through the foundation terrace deposits was probably not ideal, given that this creates confusion and the term through-seepage would more appropriately and more typically be utilized to reference seepage through the Bolivar embankment. The text of the DDR will be revised to limit the potential for future confusion related to terminology.

1-1 Backcheck Recommendation **Close Comment**.

IEPR Comment 3437987&	Geotechnical&	DDR – for ATR, Section 3.0 "Hydrology and Hydraulic Design"	12	n/a

This section states "The breach plan for Dover used an initial piping elevation of 880.0 and a trigger failure water surface elevation of 910.0 while Bolivar breach conditions used an

initial piping elevation of 910.0 and a trigger failure water surface elevation of 961.6". Comment: What was the basis for selecting these elevations?

1-0 Evaluation **For Information Only**

The information in this paragraph regarding Dover is incorrect. A failure mode of Overtopping was used in the model with a trigger elevation of 910. As Dover is a concrete dame with a calculated Imenent Failure Flood elevation of 910, these numbers are correct. With regards to Bolivar, an elevation of 905 was chosen as the final bottom elevation of the breach based on past performance of the dam and engineering judgment. (The original ground intersects the toe of the downstream slope of the dam at near this elevation. 910 was determined as the initial piping elevation based on the same basis. 961.6 was chosen as the trigger failure water surface elevation based on engineering judgment and conservative consequences estimation. This paragraph will be corrected and a basis statement added.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438005	Geotechnical	DDR – for ATR, Section 5.2 "Seepage Blanket Augmentation"	17	n/a

The end part of this section states "To further minimize future potential for residual excess uplift pressures resulting in concentrated seepage at a location of relatively thin alluvial blanket augmentation toe, the existing alluvial blanket will be penetrated via trenching along two transects during augmentation construction. This will generally consist of excavating two, 5 foot wide and approximately six foot deep trenches (or slightly greater if necessary to encounter more pervious foundation materials) that parallel the downstream toe of the existing seepage blanket. The trenches will be located ------. They will be back-filled with relatively pervious sand (less than 5% passing No. 200 sieve), thus providing potential for uniform residual head dissipation in the seepage blanket augmentation region, and then the entire overlying seepage blanket augmentation will be constructed as described above". Comment: I have the following questions regarding the seepage blanket and the trenching scheme: 1. Do the existing blanket and future augmentation of it meet the filter requirements with respect to the underlying soil? I notice that no evidence of piping within the blanket area has been found so far but is that expected to be the case for all future loadings for base condition? 2. How are these trenches expected to work differently than the blanket itself, except intercepting seepage at a slightly greater depth than the blanket base? 3. What will be the difference between the pervious sand filling the trenches and the adjacent blanket material in terms of particle sizes? Will it be significantly coarser than the adjacent blanket material? Will the pervious sand filling the trenches meet the filter criteria with respect to the blanket material around and above the trenches? 4. How will the success of these trenches be monitored? Will they have piezometers installed in them? Answers to these questions will better clarify for me the design, function, and success of the seepage blanket and the proposed transverse trenches.

1-0 Evaluation **Concurred**

In response to question 1 above, the answer is yes. In response to question 2, this is a qualitatively designed feature which should provide inherant benefits by creating a situation where residual foundation head is encouraged to dissipate more uniformly and seepage flow is encouraged to exit controlled into the blanket across a larger area as opposed to a situation where seepage blanket material would be placed on a continuous relatively impervious alluvial blanket - in this case residual head may likely be relieved at a much smaller area such as a thin alluvial blanket area or through an existing alluvial blanket defect. In response to question 3, filter characteristics of the trench-filling material will be carefully considered, and the material filling trenches will be quite similar to that used for overlying seepage berm, a difference may likely be that we will more tightly control/limit the fines content of material filling trenches to maximize drainage a bit while staying within desireable filter range. In response to question 4, the general purpose of the seepage blanket (with the few trenches) is to provide adequate seepage control for residual downstream head, by adding weight (uplift resistance) and filtered exit for emerging seepage. The effectiveness will be measured mainly by the piezometric network outlined in the instrumentation appendix to the DDR. We'll use the piezometric data of course to perform analyses such as effective stress uplift for pools that occur, and also use these data to project stability for elevated pools. If the results of these analyses/projections are favorable and other observations/data do not contradit results, then measures will be deemed successful. The DDR text in the revised version will be expanded to further clarify certain details/intent of the proposed measures.

1-1 Backcheck Recommendation **Close Comment**

The revised version of DDR should include filter criteria for seepage blanket and trenches, including grain size distributions of foundation and filter materials.

IEPR Comment 3438012	Geotechnical	General	n/a	n/a

In most of the cross-sections provided in the report, as well as in the power point presentations during the orientation briefing, the contact between the terrace deposits (gravelly fine to coarse-grained silty sand with numerous gravel zones) and the underlying silty sands (fine to coarse-grained silty sand with intermittent gravel zones), is considered to be sharp, and nearly flat and horizontal. Comment: Is this supported by the recent borehole data?

1-0 Evaluation **Concurred**

In the glacial deposits contacts are not in reality perfectly sharp as the generalized cross-sections used for discussion purposes showed. They generalized the geology for top-level discussion purposes. There is general agreement with the generalized interpretation however and recent sonic borings contained in an addendum to the DDR (note that upper portions of borings through the terrace area are gravel-rich, and lower portions of borings are not). The true variability of the glacial deposits is best understood by field observation of the exposed cut in the borrow area near the right abutment and by review of the recent sonic borings contained in the DDR.

1-1 Backcheck Recommendation **Close Comment**
Inclusion of the above statement in the revised version will be very useful.

IEPR Comment 3438022	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 2.1 "Geologic Setting"	2	n/a

The section states "Gravel strata consist of both poorly- and well-graded gravels, and are present extensively between the approximate elevations of 910 and 935 feet between Stations 5+00 and 52+50. They are directly exposed, along with underlying silty sands, in the upstream over-steepened slopes of the present Sandy Creek stream (Figure 1A) and daylight on the terrace slope downstream of the dam". Comment: No mention is made about the occurrence of these gravels on the upstream slope of the dam. Don't they daylight on the upstream face of the dam as well?

1-0 Evaluation **Concurred**
The text of the DDR will be revised to further clarify the geologic interpretation that was implied with respect to gravel strata occurrence. Gravel strata do exist beneath the blanket placed during construction along the upstream terrace, and in essence exist as the pervious upstream shell of the dam (this material came from terrace borrow area, although blending of material would have changed typical gradation to be a bit less variable than original insitu gradation).

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438029&	Geotechnical&	Appendix B: Geotechnical Analysis – Embankment, Section 2.5.2 "Piezometers"	4	n/a

After discussing the shortcomings of the piezometer network at Bolivar Dam, the last sentence of the section states "As subsequently discussed, the recommended risk reduction plan for the dam includes improvements and additions to the existing network". Comment: Does this plan include automation of all existing and additional piezometers? It is imperative that data be available from all piezometers around the year, regardless of tail

water elevation.

1-0 Evaluation **Concurred**
Agreed that frequent PZ data be acquired at all times and not just during elevated tail water conditions. The intent of the text was not to suggest that PZ data only be acquired during elevated tail water conditions, and the text will be revised to clarify this. Automation of PZs is being proposed as stated in the instrumentation addendum to the geotech DDR to assist in obtaining more frequent PZ data.

2-0 Evaluation **Concurred**
Automation of PZs is being considered as well to assist in obtaining more frequent PZ data.

2-1 Backcheck Recommendation **Close Comment**
I strongly recommend automation to the extent possible. Uninterrupted availability of monitoring data is critical.

IEPR Comment 3438037	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 2.5.2, "Relief Wells"	6	n/a

At the bottom of page 6, rehabilitation and efficiency of well W-24 are discussed. It is stated "Estimating efficiency as formation loss plus partial penetration loss, divided by total drawdown, the resulting efficiency is 85 percent, and this value is suggestive of desirable well performance potential during elevated project pools". Comment: How many of the 35 wells were tested for their efficiency? Where is the well efficiency data presented? Since it is very important to have the wells working at their highest efficiency, information about the efficiency of all wells, based on tests like W-24, would be critical. What value of well efficiency was used in the seepage studies models? Furthermore, will all wells be automated according to the rehabilitation plan? Confidence in all relief wells performing at a certain dependable level of efficiency is very important since seepage analysis and factor of safety calculations are based on relief wells working at a certain level of efficiency.

1-0 Evaluation **Concurred**
During 2009 we rehabilitated the entire relief well network at Bolivar Dam. At the time of writing the draft DDR a report summarizing the outcome and performance testing from that work had not been completed, but the general summary of the work outcome that was indicated in the DDR is still valid. At this time a rather lengthy report has been completed, and it will be referenced in the revised DDR. Two wells were specifically evaluated for efficiency via pump-testing and related analyses, and the rest of the wells condition are related to these measurements qualitatively based on consideration of similarities/differences of video-inspection, behavior during re-development, flow rates, etc. Given the expense of/time for high flow rate pump-testing and

associated analyses, it was not feasible (and not necessary in my opinion) to specifically measure efficiency in all wells directly. In practice testing a subset of project wells directly and indirectly judging approximate efficiency of other system wells is pretty typical/standard practice, and the wells to be tested can be alternated during different maintenance cycles to give a more comprehensive direct efficiency measurement of all site wells over time. So the DDR assumptions regarding decent well efficiency are still considered valid, and will be valid according to the project well system maintenance plan. The district has now a maintenance plan for the wells, and this will be discussed/referenced in the revised DDR (it generally requires frequenct assessment/confirmation of adequate well system performance potential. Not all wells are planned to be automated for flow in the DDR. Although ideal, we think we can practically make determinations of future system performance again by very close consideration of a good number of the wells via automation along with more qualitative relations of performance indicators and less frequent manual flow measurements on other wells. As an aside, the comment responder has a particular ongoing applied research interest regarding topics of relief well system design and efficiency measurement/maintenance, and this is mentioned as it should contribute to minimizing the likelihood of future well inefficiency contributing to stability problems.

1-1 Backcheck Recommendation **Open Comment**

While I agree partially with the above stated response, I don't think that a subset of two wells evaluated for efficiency is statistically representative of a sample population of 35 wells. I suggest the following for the revised version: i) provide a reference to documentation of the similarities, as indicated by video inspection, among the 35 wells; ii) specify a detailed plan for additional efficiency testing I future; iii) state average well efficiency level used in the seepage studies.

2-0 Evaluation **Concurred**

i) Reference to the final report "Relief Wells Rehabilitation Report for Beach City Dam, Bolivar Dam, and Zoar Levee Ohio Projects" by GEO Consultants will be provided. This report contains logs prepared from downhole video logging from each of the site wells and additionally contains a DVD appendix of the actual video recording for each well. ii) A detailed maintenance plan for site wells is contained in the report "Huntington District Relief Wells Maintenance Plan". This report and plan will be referenced in the DDR. The plan includes quarterly surface inspection (visual) monitoring, daily flow measurements during elevated pools, and step-drawdown testing and evaluation of all performance data on a 3-year cycle. Depending on acquired data and evaluation results, maintenance/rehabilitation activities will be performed as/when necessary to maintain desired well system performance potential. The maintenance plan specifies a sub-set of the well system for pump-testing, with the sub-set changing over time such to sample a larger percentage of the wells in the system. The responder is of the opinion that this approach is sufficient for accomplishing plan objectives, and notes that perfectly ideal plans which are not optimized for practicality and ignore budgetary concerns are much less likely to be implementable. So the definition of an effective maintenance plan here is one that can accomplish the objective but is cost-effective enough at the same time to allow it to actually be implemented. iii) Well efficiency consideration during seepage analyses is not a well established process in the literature and is a challenging topic which is continually being researched. For the bolivar report the topic was addressed by treating the developed filter/outwash region surrounding the well screen as a random variable with a (somewhat conservative perhaps) coefficient of variation of 90% applied to this foundation region's hydraulic conductivity. Seepage models were performed with expected, and +/-1 standard deviation values to determine (along with other random variables) the reliability or affect on factor of safety distribution that uncertainty in well performance potential has. The responder has developed and incorporated iterative (theoretical vs. actual drawdown) well efficiency methodology for other situations better suited by blanket theory, but in this case the approach taken, and seepage modeling calibrated to current well condition indicates the wells are at a desired level of performance currently. So the objective from this point forward, or the goal of the maintenance plan for the system is to ensure that efficiency is maintained within +/- 10% of current level.

2-1 Backcheck Recommendation **Close Comment**
I am neither suggesting an ideal nor an impractical plan, and I am very aware of the economic constraints. However, any remediation plans need to be thorough, scientifically sound, and based on the best compromise between economy and safety, with the safety coming first. The plan should ensure that the problems will not arise again so that the money does not have to be spent fixing the same problem multiple times. That, in my opinion, is a better way to economize. The above explanation answers my concerns.

IEPR Comment 3438048	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 4.4.1, "Embankment Materials"	10	n/a

This section states "Regarding the potential for through seepage concerns associated with closure section construction, it is noted that the transition between clayey sand and silty clay core materials near Station 59+00 occurs between the approximate elevations of 880 to 940 feet. The pool has been in and above this elevation range a number of times in the past without any signs of through seepage or related erosion observed". Comment: Although seepage and piping have not been observed in this area in the past, I believe the potential for its occurrence can not be completely ruled out under higher loading conditions.

However, the construction of a seepage barrier should address any concerns in this regard.

1-0 Evaluation **Concurred**

Potential failure modes related e.g. to staged construction is being considered as part of potential failure mode and risk assessment analyses being performed subsequent to the draft DDR preparation. Relevant findings of these investigations will be reported in the DDR. Agree that the proposed seepage barrier should likely resolve any such potential concerns.

1-1 Backcheck Recommendation **Close Comment**

The response to my comment is not very clear at this stage but I hope the provision of relevant findings in the revised version will add to clarification.

IEPR Comment 3438054	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 5.1.1, "Modeling Overview"	14 (top)	n/a

"Seepage model geometries and conductivities were adjusted to produce calibrated models with results that closely matched piezometric levels (average difference of <= 1.5 feet), relief well flows [average difference of < 0.5 gallons per minute per foot (gpm/ft)], and uncontrolled seepage quantities observed versus location during these previous events". Comment: What was the magnitude of conductivity values compared to those listed in Figure 5? What is the basis for selecting the acceptable differences between modeled values and those observed in the field? Is that based on judgment or established guidelines by USACE?

1-0 Evaluation **Concurred**

The level of calibration achieved and deemed adequate in this modeling work is based on judgement of the modeler. The level of calibration in terms of agreement with measured piezometric reponses, observed seepage flows, and measured relief well flows is reported relative to 3 previously experienced pools at the project. It is noted that in general, mismatches between modeled and measured piezometric elevations are felt to occur in conservative fashion, as generally when they occur they deliberately occur such that modeled pz elevations miss measured pz elevations on the high side. The statistics for conductivitiy values in table 5 were developed by varying model conductivities - conductivity values were varied and when good calibration of the models did not occur then these points were determined as the upper and lower bounds for the conductivity value being evaluated. So in summary, the answers to the questions are that the range of conductivity values are indicated by the statistics in Figure 5 which are based on model sensitivity analyses and regional knowledge of material properties, and the level of calibration was based on what the modeler deemed adequate. Specific calibration guidelines were not utilized in this decision, but the modeler is confident that the calibration results reported in the DDR should fair well/adequate if compared to any particular calibration guidelines that the commenter may have in mind (the responder does not know what particular guidelines the reviewer may be referring to, but would be interested to know).

1-1 Backcheck Recommendation **Close Comment**

My comment is of general nature and does not refer to any particular guidelines.

IEPR Comment 3438061	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 5.1.2, "Model Safety Predictions", p. 16 and Figure 6 (p. 38)	p. 16, p. 38	n/a

Comment: Figure 6 is referred to in Section 5.1.2. I have the following questions concerning Figure 6 (and all other similar figures pertaining to base condition as well as after rehabilitation condition): 1. How are the maximum values of vertical and horizontal gradients, Iv max and Ih max, computed by the model? In the figure, these values are given as 1.92 and 0.60, respectively. If I use the modeled head (equipotential lines) and the vertical scale given in the figure, I do not get a vertical gradient of 1.92. Similarly, using the horizontal scale and a total head loss of 52 feet across the system, I get horizontal gradient values of approximately 0.12 considering the upstream blanket and 0.35 ignoring the upstream blanket. These values are significantly lower than Ih max of 0.60. 2. Did the model consider the upstream blanket or ignore it while computing Iv max and Ih max? 3. The equation for calculating FS(effective uplift), as given in the text [(?b x blanket thickness)/(total head at blanket base – tail water elevation)], compares the effective stress at the blanket base with the difference in upward and downward water pressures at the blanket base. In case of Figure 6, it leads to a FS value of 55.38, as indicated in the figure [i.e., (120 – 62.4) (6)/(0.5) (62.4)   55.38]. However by considering effective stress in the numerator of the equation, isn't the equation taking into account the buoyancy effect of water twice? What would a FS(effective uplift) equal to 1 mean in this case? 4. How is FS(piping) D/S of blanket calculated? Neither are the exit gradient values D/S of seepage blanket given nor are the vertical and horizontal critical gradient values (Icv and Ich) given for the material D/S of the blanket. Furthermore, a FS(piping) value of 32.97 (Figure 6) implies an extremely gentle hydraulic gradient. Is this value consistent with the piezometric data?

1-0 Evaluation **Concurred**

question 1: DDR will be revised to clarify manner of gradient calculation and FS calculation. The modeled gradient used is that in the emerging seepage/exit region, not average gradient considering net head and dissipation along total flow path. FS calculation for erosion initiation consider exit gradient and critical gradient for the material. The horizontal critical gradient is less than the veritical critical gradient, dependent upon friction angle of material. The exit gradients/uplift pressures used in computations for piping and effective-uplift, are not readily inferable from the output figure (e.g. figure 6) because of the scale at which it is presented. In the model (on computer screen) it is possible to enlarge the model and contour head dissipation versus distance at a much more localized exit/emerging seepage position. So the figures show the large scale head distribution, but smaller-scale head distribution at erosion initiation points are not readily inferable from figures. Question 2, I think response to question 1 probably clarified this, but the upstream blanket is considered by the model and affects overall subsurface head distribution, and therefore exit gradients/uplift pressures. E.g., if I had low permeability blanket extending to infinity upstream, this would have the affect of reducing head in the project foundation and lowering exit gradients. Question 3, the effective and total stress formulations for evaluating uplift stability yield equivalent results at a FS value of 1.0, and the effective stress results deviate from 1.0 more quickly than the total stress. Effective stress formulation is more applicable for this situation while total stress is more applicable for concrete structures. The commenter could check the FS value in a different way, by considering that the Pmax value of 30.10 (in figure 6) give an excess head of .10 feet (value pz would read above tail water). Since the downstream blanket is 6 feet thick, this gives an average gradient through blanket of 0.0167. For critical gradient of the blanket of 0.92, a piping factor of safety calculation (0.92/0.0167) gives similar FS value of 55. The effective stress formulation for uplift is generally comparable for most situations, not all, to the results of a piping FS. So this is another way to consider the condition being discussed here - by looking at the average gradient through the blanket and considering if this is of concern. Question 4, the saturated weight of the materials being considered for erosion is 120 pcf, so critical vertical gradient is 0.92 (DDR text will be revised to clarify this). Modeling results are the best that can be predicted based on available pz data. We have good model calibration up to the pool of record, and beyond that estimates of gradients are subject to typical limitations of trying to project head distributions for loading events not yet seen. The tail water for this event is projected as 903 +/- 10 ft.

1-1 Backcheck Recommendation **Open Comment**

1. How did the modeled gradient pick emerging seepage/exit region? Was this based on actual seepage points? 2. My comment (# 3) regarding F.S. is not about the effective stress vs. total stress approaches. I am concerned about buoyancy effect being considered twice. This point needs further discussion I like to be educated about the basis aspect of calculations). 3. The comment regarding the agreement between a F.S. 32.97 and piezometric data (# 4) has not been addressed.

2-0 Evaluation **For Information Only**

1. The factor of safety values reported represent the minimum value

determined for a particular region of the model. A number of gradients would have been measured for example in the region of the downstream blanket, but the factor of safety for the location having the most adverse combination of uplift and resisting force is the value reported. Generally, where seepage has been observed at the project is where the model results indicated the lowest factor of safety values. 2. The effective stress uplift or sometimes called gradient method approach is widely used and cited in a number of publications. One thing not clear in many publications is how to define excess head. Many publications represent it as piezometric level minus top of ground, however, the correct way to calculate this, and the way it was done for this study was to calculate excess head as piezometric elevation minus the top of ground elevation or tail water elevation - whichever of these two is greater. In the book "Soil Mechanics in Practice" by Terzaghi and Peck (1948, pg. 54) there is an equation showing how effective stress (thickness*buoyant weight) is decreased by seepage force (gradient*water weight*thickness). As soon as the gradient in this equation becomes equal to the critical gradient then the effective stress becomes zero. So the equation used in the DDR report to calculate effective stress uplift is the same as this equation. In the DDR the factor of safety equation numerator is the same (thickness*buoyant weight), while the denominator is excess head*water weight. The excess head*water weight in the denominator is equivalent to Terzaghi and Peck s gradient*water weight*thickness, the two denominators are just stated differently. E.g., I can replace gradient term in Terzaghi and Pecks formulation with excess head divided by thickness, which gives me a denominator equivalent to excess head/thickness*water weight*thickness. Then the 2 thickness terms cancel out, and I m left with the denominator of excess head * water weight. 3. The FS 32.97 results from the critical gradient (icv) divided by the measured gradient of about 0.03; both of these are reported on the referenced plot. The agreement between this FS prediction and piezometric data is unconfirmed for the modeled pool elevation as the pool in this model is 30 ft higher than the project has previously experienced. Based on the good model calibration as discussed in the DDR text, and as demonstrated on model plots for previously experienced pools of 936, 949, and 952 ft (i.e. compare observed vs. predicted piezometric elevations), the responder feels that this factor of safety agrees as closely as it could with available pz data acquired at the site.

2-1 Backcheck Recommendation **Open Comment**
The above explanation does not answer my question number 1. I understand the FS values reported are for the worst conditions. My question is regarding the seepage areas identified by the model. Do the seepage areas identified by the model match the seepage areas observed in the field? Do the hydraulic gradients computed by the model match the hydraulic gradients indicated by the piezometric data? Do the FS against piping values computed by the model match the areas where piping has been noticed in the field, i.e. do the potential piping areas identified by the model match the actual piping occurrences? I am a little

concerned about the very large values of FS, especially when I am told that the figures in the DDR report are not true representations of the model output. The term "extra head" or "excess head" clarifies the situation with respect the equation regarding the quick condition. I'll check the calculations again.

3-0 Evaluation **Concurred**

The seepage areas identified by the models do agree with past field observances. The DDR text will be expanded to further discuss specific modeling results and field observance agreement. Personnel performing modeling for Bolivar have observed seepage and erosion initiation first hand at the project during many elevated pool events over the past decade. So, in addition to calibrating against site instrumentation data, calibration of models against prior locations of seepage and erosion initiation occurred during model development. Above the pool of record (elev. 952), modeling results represent unconfirmed projections of piezometric levels, seepage locations/quantities, and erosion initiation potential. With regard to the high FS values reported by modeling in some instances, this results from the elevated tail water levels projected by hydraulic models for lower frequency loading events, and due to the effective stress calculation approach (which yields ever increasing or decreasing FS values the further away from a solution of 1.0 that you get). The text of the DDR will be expanded to further explain the reasons for the high FS values that are presented. The text will also be expanded to indicate how reliability analyses and engineering judgement have been and will continue to be employed so that a possibly unconservative path forward will not occur with regard to high projected tail water elevations for low frequency events. That is, the final remedial actions taken at the project will provide tolerable stability against expected tail water elevations, but also provide tolerable stability against the much less likely condition of an elevated pool coupled with a low tail water elevation. Although unexpected based on hydraulic models, the remedial actions designed and implemented will be evalutated to ensure stability for such a situation in case it does ever occur at some point in the future.

3-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438069	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 5.1.2, "Model Safety Predictions"	16 (top)	n/a

The text here states "In the terrace slope area and beyond the downstream blanket, the piping factor of safety [FS(piping)] was calculated as the minimum value obtained by dividing maximum vertical and horizontal gradients (Iv max and Ih max) by the respective

critical gradients (Icv and Ich) for vertical flow [---------] and horizontal flow [--------]". Comment: Shouldn't this be backwards, i.e. critical hydraulic gradient divided by exit hydraulic gradient?

1-0 Evaluation **Concurred**

Yes, this was an unfortunate typo in the text. This will be corrected in the revised DDR.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438075	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 5.1.2,	16	n/a

At the bottom of page 16, it is stated "Considering medium- to coarse-grained sand and the Bolivar structure length near stations 49+00 and 51+00, the critical gradient required for a pipe to move half way to the water source was calculated for example as 15% greater than that required to initiate erosion. Given this, FS(progressive erosion) values were therefore figured as 115% of the minimum FS values for erosion initiation". Comment: The rationale for increasing the critical gradient values by 15% is not clear from the above stated text. Does it mean that a critical gradient of 1 was increased to 1.15? The above statement needs clarification.

1-0 Evaluation **Concurred**

Yes, this is generally what was meant by the text. In this case, the critical gradient was modified to represent the likelihood of not only erosion initiation at the exit point, but also an advanced condition where the pipe would have the capability to progress to make connection with the reservoir. The text will be revised to provide further clarification on what exactly was done here and further rationale/reference to research supporting this type of calculation.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment& 3438083&	Geotechnical&	Appendix B: Geotechnical Analysis – Embankment, Section 5.2.1, "Probabilities of Unsatisfactory Performance"	17	n/a

Pr(u) represents the probability that FS(progressive erosion) is less than the limit state (i.e. FS of one), while ? is a relative measure of current condition and provides a qualitative estimate of expected performance". Review Comment: Is there an acceptable value of ?

used for the project as the limiting value?

1-0 Evaluation **Concurred**

There is not an established acceptable value of Beta, i.e. guidelines which state that design should occur to certain Beta value. It is a value that means something relevant to one particular loading, and another parameter that serves as a basis for improving situational understanding by the analyzer/reviewer. So what is a good/arguably acceptable Beta value may be different to different people, and in different situations. I think I can present a Beta value to support an opinion I have for instance, but it is not the only piece of evidence that must be considered, and arguably not the most important (e.g. not as important for decision-making as a calculated annual probability of failure results for a particular failure mode being discussed). The text of the DDR will be revised to make sure that the reader is not interpreting the writer as inferring that a certain Beta value by itself constitutes acceptable/unacceptable situation.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438099	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 6.1.1, "Seepage barrier Construction"	22-24	n/a

The section refers to Figure 15 (p. 47). I have similar questions regarding Figure 15 as Figure 6. At its end, the section states "Therefore, and also considering the potential for increased seepage gradients at the lateral end of a seepage barrier, the recommended barrier layout extends northward through Station 27+00 to Station 20+00, where similar 3D-related affects and concerns do not exist, and where analyses and judgment indicate desirable through seepage and slope stability FS and Pr(u) values exist without a barrier". Comment: There are many case histories in dam engineering where the contact zones between concrete and soil structures, especially the lateral contacts (such as the contact between a concrete dam and an embankment), happen to be the zones most susceptible to seepage and piping. Although the analyses show the embankment to be stable under all loading conditions north of Station 20+00, in the interest of redundancy, resiliency, and robustness, I suggest that additional analyses be performed, including a cost-benefit analysis, for a full-length partial-depth seepage barrier.. This will provide a more uniform seepage barrier along the entire length of the embankment. I also think that the choice of a partial-depth seepage barrier should be examined in light of the experiences with such barriers at other projects. In case of an extreme event, upward directed seepage forces on the downstream side of the seepage barrier can create unforeseen piping related problems. It was the economic constraints that resulted in the use of a partial-depth diaphragm wall, instead of the initially designed full-depth wall, at the Wolf Creek dam but that decision turned out to be very costly, as is clear from the current, ongoing rehabilitation measures. I

realize the geologic conditions at the Wolf Creek dam site are different than the Bolivar dam site and that high level of tail water at the Bolivar dam site, in case of large flood events, is actually helpful against piping problems, but I believe we should take another look at the partial-length and partial-depth seepage barrier in light of past experiences at other projects rather than relying entirely on model results. As stated previously, we should consider, at least, the option of using a full-length, partial-depth seepage barrier.

1-0 Evaluation **Concurred**

I believe that the referenced case histories quite often involve a bit of a different situation in which e.g. poor compaction during original construction causes subsequent difficulty. Nonetheless, the point is relevant and well taken, and this issue is receiving further consideration as part of potential failure mode and risk assessment analyses being performed for Bolivar Dam currently. This work will take a look at the proposed remediation within the context of a different set of design guidelines and risk considerations (such as the commenter recommends) than were utilized for this MRR/DDR work to date (more heavily focussed on economic justification). The findings of these studies may have an impact on the proposed geometry of the seepage barrier (i.e. lateral extent and depth), and the DDR and planned construction will be modified accordingly.

1-1 Backcheck Recommendation **Open Comment**

I would like to see the results of new analyses. This, in my opinion, is the most critical issue of the rehabilitation plan.

2-0 Evaluation **For Information Only**

The referenced issue evaluation study for Bolivar Dam is currently in progress. If the reviewer has reached any conclusions or has developed any specific opinions regarding the appropriateness of the proposed remediation, as it is presented in the DDR, largely with respect to economic justification, then such comments would be potentially useful and welcome at this time.

2-1 Backcheck Recommendation **Open Comment**

My comment is based on my reading of the DDR. My opinion is that partial cut offs can be risky because of the unpredictable hydrologic conditions that may develop during the service life of a project. Our best guide is our past experiences. There are a number of case histories of poor performance of partial cut offs. I am cognizant of the economic constraints but we need to put safety first and not ignore our experiences because that turns out to be more expensive in the long run.

3-0 Evaluation **Concurred**

Agree that partial cutoffs are risky relative to full cutoffs. The DDR was prepared under Major Rehabilition Program guidelines, which are heavily focussed on economic aspects and cost-justification of remedial work. In the DDR case, full-length and full-depth cutoff was was not economically justified. Since development of the DDR however, the Bolivar project is now subject to guidelines contained in ER 1110-2-1156 (Safety of Dams - Policies and Procedures). The 1156 regulation places it s main emphasis on achieving tolerable life safety risk, with less importance placed on economic aspects. So under this guidance economic justification for a full-depth cutoff wall for example, will not necessarily be required. There is a study already in progress for Bolivar, being performed by a multi-disciplinary and multi-district cadre, which is focussed on development of a remedial design to meet life-safety risk guidelines as required by the 1156 regulation. This study is likely to be completed late 2011, and will then undergo extensive review, with life safety guidelines being shown to be met before construction of the final remedy for Bolivar Dam. So, in summary, the remedial design for the project is now being evaluated under a different set of guidelines than it was for the DDR, and through this process, the potential need for and benefits provided by a full-depth and full-length cutoff wall will be thoroughly evaluated. The final design to be constructed for Bolivar will be that which provides a tolerable life-safety risk. Note that the 1156 guidance (if you search online for it) is not the original version of the regulation published in the 1990s, but a document that has just recently been entirely re-written, finalized and approved by USACE headquarters in late 2010 to early 2011.

3-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438105	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 6.1.3, "Relief Well Efficiency Maintenance"	25	n/a

The last sentence of this section states "Without benefits provided by the well system and the above-discussed seepage blanket augmentation, a fully (foundation) penetrating seepage barrier would be recommended between Stations 20+00 and 64+00 to provide adequate geotechnical reliability". Comment: This statement shows that the success of a partial-length and partial-depth seepage barrier is highly dependent on fully functioning relief well system and seepage blanket. The degree to which one can depend on the long-term performance of a fully penetrating seepage barrier far exceeds the long term performance of a relief well system or a seepage blanket. In my opinion, properly functioning relief wells and seepage blanket should add redundancy and resilience to the seepage barrier instead of controlling its degree of success. What is the expected level of efficiency of the relief well system (85%, 80%, etc.) implied in the above statement? This statement supports the need for looking into the feasibility of a full-length and partial-depth seepage barrier, at the minimum.

1-0 Evaluation **Concurred**

Agreed that the long-term performance of properly installed barrerier has more certainty than properly installed well system. The seepage blanket and well system do provide certain benefits, and therefore if they were removed or become inoperable the recommended plan would provide less overall project reliability. The expectation for well system performance in the proposed remedial plan is that the wells will continue to perform well in the future. This will require continued following of the District's well system maintenance plan for the project, and at this point the plan is to do so (i.e. maintain wells, and if necessary replace any wells determined to be problematic/irrecoverable efficiency loss). If the potential for wells to lose efficiency was a basis for automatically excluding them as a remedial option, then they exist anywhere in the country (i.e., the point I m trying to make is that along with the disadvantages they do have some benefits/advantages). We are continuing work through risk analyses being performed subsequent to this draft DDR publication to more thoroughly evaluate the proposed permanent risk reduction measures for the project. One of the things that will be looked at more closely during this ongoing work is the suggested full-length and partial-depth seepage barrier.

1-1 Backcheck Recommendation **Close Comment**
I am not suggesting we should exclude/ignore the benefits of a well-maintained, efficiently performing relief well system. I am trying to emphasize the differences in long-term reliability of various components of the rehabilitation plan.

IEPR Comment 3438111	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 6.1.4, "Instrumentation System Improvements and Automation"	25	n/a

The section states "New piezometers (e.g. nested depth sets) will be installed at various locations through and beyond the proposed downstream seepage blanket augmentation to monitor head values and gradients". Comment: How many piezometers will be installed beyond the seepage blanket and what will be the basis for their locations?

1-0 Evaluation **Concurred**
The details concerning locations and number of additional proposed piezometers are contained in the Instrumentation addendum to the Geotechnical appendix of the DDR. The basis for the number, locations, and depths was to install a network that should provide a good distribution of subsurface head distribution across the project. Any specific thoughts as to whether the proposed network seems adequate for this purpose would be quite welcome, as it would assist in accomplishing this goal. Of course we d like a piezometer everywhere at the project, so in developing the proposed plan we tried to counter this desire by considering practical/cost-related aspects.

1-1 Backcheck Recommendation **Close Comment**
I'll re-examine the instrumentation addendum.

IEPR Comment 3438117	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 6.1.4, "Deterministic and Probabilistic Analysis Results: Recommended Risk Reduction Plan"	27	n/a

"Deterministic through seepage FS(progressive erosion) for this load case has been reduced from less than 0.6 (base condition) to greater than 2.9 as a result of the seepage barrier". Comment: Should the word "reduced" be changed to "increased"?

1-0 Evaluation **Concurred**
Yes, this was an unfortunate typo in the report. Thanks for catching this, it will be corrected in the revised version of the DDR.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438128	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 6.2.1, "Total Embankment Summary: Recommended Risk reduction Plan"	27	n/a

This section states "Residual low Pr(u) values do still exist in the with-rehabilitation condition for under seepage and through seepage at certain pool elevations and project reaches (Stations 57+00 and 51+00, respectively); these residual Pr(u) values have associated Beta values which are near 3.5". Comment: The implications of this statement are not clear from the text provided in the section. I assume the residuals are insignificant. What is the acceptable range of Beta values? I need additional clarification regarding the above statement to be able to make a definite comment about it.

1-0 Evaluation **Concurred**
The DDR will be revised to provide further clarification as to the

intent/implications of the results that are referenced. What the text generally meant to say was that with the proposed plan we ll still have residual probabilities of erosion occurring, but they are low, and the annual probability of failure related to them is low and deemed acceptable with respect to guidelines as well. I.e., we don t think we need to, nor could we even if we tried as hard as we could, reduce probabilities of all undesireable events to absolute zero. So the text meant to convey these types of opinions/conclusions, and it will be revised to do a better job of this.

1-1 Backcheck Recommendation **Close Comment**
The above response clarifies significantly the statement pertaining to Beta value included in the first draft. Addition of this clarification to the final draft will be very helpful.

IEPR Comment 3438136	Geotechnical	Appendix B: Geotechnical Analysis – Embankment, Section 7.1, "Summary of Analyses Results"	29	n/a

"Base condition analyses performed for Station 5+00 located near the right abutment of the dam and Station 20+00 indicate that the embankment is stable for all loading conditions for seepage and slope stability. Therefore no risk reduction measures are recommended for this reach. Surveillance and instrumentation monitoring will be performed during future high pools to ensure adequate performance of this reach". Comment: The potential for inadequate performance within this reach can not be completely ruled out, especially with the seepage barrier terminating at Station 20+00. The feasibility of extending the barrier through this reach should be evaluated.

1-0 Evaluation **Concurred**
Agree with the comment, and as stated in previous comments the suggested considerations are being made as part of ongoing risk analyses work subsequent to publication of this DDR draft. The findings from the risk analysis work may alter the geometry of the proposed seepage barrier, and the DDR and construction plans will be revised accordingly.

1-1 Backcheck Recommendation **Open Comment**
I would like to read the findings of the additional risk analysis report before closing this comment.

2-0 Evaluation **For Information Only**
The referenced issue evaluation study for Bolivar Dam is currently in progress. If the reviewer has reached any conclusions or has developed any specific opinions regarding the appropriateness of the proposed remediation, as it is presented in the DDR, largely with respect to economic justification, then such comments would be potentially useful and welcome at this time.

2-1 Backcheck Recommendation **Open Comment**
My response is the same as for comment # ID 3438099

3-0 Evaluation **Concurred**
Note, this response is similar to that presented for comment #3438099, with a few minor differences. The DDR was prepared under Major Rehabilition Program guidelines, which are heavily focussed on economic aspects and cost-justification of remedial work. In the DDR case, full-length (past station 20+00) cutoff was was not economically justified. Since development of the DDR however, the Bolivar project is now subject to guidelines contained in ER 1110-2-1156 (Safety of Dams - Policies and Procedures). The 1156 regulation places it s main emphasis on achieving tolerable life safety risk, with less importance placed on economic aspects. So under this guidance economic justification for a full-length cutoff wall for example, will not necessarily be required. There is a study already in progress for Bolivar, being performed by a multi-disciplinary and multi-district cadre, which is focussed on development of a remedial design to meet life-safety risk guidelines as required by the 1156 regulation. This study is likely to be completed late 2011, and will then undergo extensive review, with life safety guidelines being shown to be met before construction of the final remedy for Bolivar Dam. So, in summary, the remedial design for the project is now being evaluated under a different set of guidelines than it was for the DDR, and through this process, the potential need for and benefits provided by a cutoff wall past station 20+00, any other potential remedial alternatives past station 20+00, will be thoroughly evaluated. Uncertainty and our inability to perfectly project seepage conditions in certain project reaches such as station 20+00 to the right abutment, will be identified, and appropriate cautionary design measures will be taken to ensure appopriate level of conservatism is employed for all project reaches to guard against seepage-related failure during extreme loading conditions. The final design to be constructed for Bolivar will be that which provides a tolerable life-safety risk.

3-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438153	Geotechnical	Appendix C: Geotechnical Analysis – Abutment, Sections 2.2 (Left Abutment Geology), 4.1.4 [Design Documentation Report Explorations (2009-2010)], 5.0 (Subsurface	5-10	n/a

Conditions), and
5.1 (Hydraulic
Pressure Testing)

On page 5 under "Left Abutment Geology", the text states "At the base of the Lower Mercer Limestone is a gray, soft to moderately hard underclay. The base of the limestone and this clay seam are located at the elevation in which seepage water has been observed during high pools. Notable amounts of core loss were noted within the limestone and clay units in the 2006 borings. The elevation of this clay seam is at approximately 939, which coincides with the pool elevation in which the seepage appears". At the bottom of page 8 under "Design Documentation Report Explorations (2009-2010)", it is stated "The borings were also instrumented with open tube piezometers to allow for monitoring of the seepage within and below the Lower Mercer Limestone at approximate elevation 938. Under "Subsurface Conditions" on page 9, the 3rd paragraph states "A loss of 0.2' was noted directly below the Lower Mercer Limestone, within a clay seam at approximately 52.7' (el. 934.8), which is the suspected seepage zone". On the same page, 4th paragraph states "Flow within the suspected zone of seepage within the underclay (~ depth of 57 to 58 feet) (~el. 929 to 928) was 0.76 cfm (23 Lugeons)". Under "Hydraulic Pressure Testing", page 10, the second paragraph states "Flows as high as 2.66 cfm (~120 Lugeons) were measured from elevation 932.9-941.6, which is the location of the Lower Mercer Limestone and underlying claystone (the Middle Mercer Coal was not present in this boring)". On the same page, the 3rd paragraph mentions "A total of 15 pressure tests in boring C-09-33 showed water takes from 1.8 to 2.1 cfm (69 to 85 Lugeons) from elevation 933.7-925.0 which is the location of the bottom of the Lower Mercer Limestone (including a 1.1 foot void), Middle Mercer Coal and underlying claystone and shale units". Finally, the 4th paragraph, on page 10, states "Flows ranging from 1.54 to 1.96 cfm (~ 69 to 92 Lugeons) was measured from elevation 946.9 to 933.9 which is the location of a "Middle Shale" unit. This zone includes a 1.0' thick, soft clay underlain by a zone of 0.5' loss and the Lower Mercer Limestone". Comment: From the above statements it is clear to me that the seepage occurs within a zone involving the Lower Mercer Limestone and the underlying coal and underclay units, approximately 7 foot thick. However, what is not clear is if this zone, especially the bottom of the Lower Mercer Limestone, occurs at variable elevation. Figures 1 and 2 (page 11), showing hydraulic pressure testing results, indicate maximum flows at an approximate elevation of 940. Field observations during past high pools show seepage levels at approximately 935 feet elevation (DDR Report for ATR, p. 9). Is the base of Lower Mercer Limestone encountered at variable elevations?

1-0 Evaluation **Concurred**

While looking into the issues brought up by this comment, it was discovered that some of the elevations on the boring logs were incorrectly converted from depths and 30 degree batter. These boring logs are currently being corrected. Once these corrections are complete, the text in the DDR will be corrected accordingly. Some discrepancies can be attributed to loss during coring and assumptions made as to the exact locations of that loss. There is only an approximate 4 degree dip in the upstream direction. In reality the elevation of the limestone does not vary by more than a couple tenths of a foot.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438155	Geotechnical	Appendix C: Geotechnical Analysis – Abutment, Section 5.2.3	16&	n/a

Comment: The shale comprising unit 5 is tested but shale from units 9 and 10 is not tested? Is there a specific reason for selecting some units for lab testing and not the other? Further, no results are reported about any of the claystone samples. Is that because of the lack of sufficient sample?

1-0 Evaluation **Concurred**

The objective of the testing was to provide prospective contractors with the excavatability of the rock units in the abutment. Unit 9 is the founding unit for the proposed cutoff wall and unit 10 is below the founding elevation for the proposed cutoff wall. There was very little sample material available from unit 9 and it is assumed that it would not be problematic to excavate in comparison to some of the other units tested. In regard to the claystone, there was little to no material that could be sampled and tested due to the high amount of weathering and fracturing in this unit. Further, it is assumed that it can be easily excavated.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438160&	Geotechnical&	Appendix C: Geotechnical Analysis – Abutment, Section 6.2, "Construction Methodology"	20	n/a

The section states "As the seepage barrier extends southwest from Station 59+00 toward Station 64+00 (where it will further extend southwest to Station 66+00 entirely in rock, and then head west to its downstream termination), the main embankment seepage barrier will be a fully penetrating feature; i.e., the entire soil foundation beneath the embankment will be cut off". Comment: What will be the total length of the left abutment seepage barrier and what is this length based on? I did not find this information in Appendix C. Does the lower Mercer Limestone outcrop beyond the termination point of the seepage barrier? If so, would the barrier ensure that there would be no seepage beyond its lateral extent?

1-0 Evaluation **Concurred**

The length of the abutment cutoff wall starting at station 64+00 is approximately 360 feet. This length is based on transitioning from the embankment into the abutment and extending at a right angle to the outside of the tunnels and then transitioning slightly downstream. The limestone does outcrop on the outside of this wall in the spillway; however, it is covered by concrete side walls in the spillway.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438165	Geotechnical	Appendix C: Geotechnical Analysis – Abutment, Section 6.2, "Construction Methodology"	21	n/a

The first paragraph on page 21 states "For this report it is estimated that the cut off wall panels will be ten feet wide with two foot overlap between panels". Reviewer Comment: Does a 2-foot overlap mean a 1-foot overlap on each of the two sides of each panel or does it mean 2-foot overlap of each side? In other words, in each 10-foot wide panel, will the portion that is not overlapped be 6 feet wide or 8 feet wide?

1-0 Evaluation **Concurred**

It means 1 foot overlap into an adjacent panel, thus the portion that will not be overlapped will be 8 feet.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438171&	Geotechnical&	Appendix C: Geotechnical Analysis – Abutment, Section 6.2, "Construction Methodology"	21	n/a

The second paragraph on page 21 states "The radial grouting will be performed along three lines of tunnel stationing, with one falling within the cut off wall, one upstream of the cut off wall and one downstream of the cut off wall. The grout holes will be drilled every 45 degrees along the entire circrumference of the tunnel". Comment: 1. What will be the depth of radial grout holes? 2. The 45 degree angle-will result in 8 holes along the circumference. Will this spacing be enough to seal all the fractures ("windows")? How will the success of grouting be verified? 3. The base of the cut off wall will be at elevation 915 above the tunnels and the tunnel crest is at elevation 895. Is the purpose of grouting within the plane of the cut off wall to seal the 20 ft interval between the tunnel crest and the base of the cut off?

1-0 Evaluation **Concurred**

The purpose of the radial grouting around the tunnels is to seal the gap between the tunnels and the cutoffwall and to prevent any potential seepage along the tunnels. It is assumed that a grout hole every forty-five degrees will be affective in sealing any "windows ; however, if high grout takes are encountered, subsequent split spaced grount holes could be included between these primary holes until tight holes with low grout takes are obtained. The text will be revised to include this verbiage about subsequent split spaced holes.

1-1 Backcheck Recommendation **Open Comment**

The response answers parts 2 and 3 of the comment but not part 1. May

be I am not able to picture the grouting plan very well.

2-0 Evaluation **Concurred**

The depth of the grout holes is currently set at 20 feet; however, if, during grouting operations, high takes are encountered in that zone, the holes will be deepened accordingly.

2-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3438175	Geotechnical	General	n/a	n/a

General Comment/Question: During the orientation briefing we were shown areas where vegetation had been cleared off from embankment slopes and areas downstream of drainage blanket. Will selected areas downstream of the augmented drainage blanket (i.e. after rehabilitation), that are considered critical with respect to piping (where sand boils have been observed in the past), be also cleared of vegetation so that they can be monitored for any piping activity in addition to piezometric data from such locations? This will be important to assure the success of the various rehabilitation components.

1-0 Evaluation **Concurred**

Additional clearing is planned from approximately dam centerline station 40+00 to 60+00 downstream of the existing seepage blanket to within 30 feet of the old Sandy Creek channel so that the area can be monitored. See drawing C-1 for current clearing limits. A portion of this area has been designated as wetland requiring mitigation which the Corps is pursuing. Additional clearing may also be recommended pending results from an on-going risk assessment of the project.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3441042	Other	Addendum H Drawing I-101	Inclinometer ID Table	Instrument Elevations

Elevations listed imply 20-foot gage lengths for In-Place Inclinometers (IPIs). Reevaluate if the accuracy for this configuration is adequate. Reevaluate if installation curvatures and casing diameter can accommodate 20-foot gages without interfering with the inside of the casing wall. If either turns out to be compromised, the data for small displacements of 0.1 inch per 20 feet or less will turn out inconclusive or useless. Consider 10-foot gages max.

1-0 Evaluation **Concurred**

The DDR Design did not intend to imply that the IPI rod lengths would be 20 ft. It was intended that the rod segments be 10 ft in length. We have clarified the detail on Drawing I113 and have added a note to the table on Drawing I101 showing the IPI sensor elevations. Please note that not every rod segment will have an IPI sensor. We believe that six sensors per IPI location can provide sufficient coverage to indicate if unexpected lateral movement is occurring. If movement indicated by the IPIs warrants that more detailed readings be made, the plan is to remove the IPI string and perform manual profiling on a 2 ft interval until the wall panels are completed in a given area.

1-1 Backcheck Recommendation **Open Comment**
This method could work, although removing the IPI's for manual surveys, even at 50% coverage is not the best the solution, in my opinion. Drawing I113 implies that inclinometer casing and piezometers are combined and grouted into the boring. I would suggest that the other piezometer boring detail that just indicates a tremie pipe also include an inclinometer casing for times that manual surveys need to verify what the IPI-array might be suggesting. Please comment back.

2-0 Evaluation **Concurred**
Inclinometer casing can be added for the previous piezometer only installations near the IPI instruments as suggested. The new inclinometer casings will be for manual read purposes. This will provide some redundancy with nominal cost impacts.

2-1 Backcheck Recommendation **Close Comment**
I recommend that inclinometer casings for manual readings be provided as indicated above.

IEPR Comment 3441046	Other	Addendum H Table 3-2	n/a	n/a

The strengths listed are true for the traversing (manual) inclinometers, but not necessarily for in-place sensors, particularly for gages at 20-foot lengths. A 10-fold or greater loss of accuracy is lost relative to the manual system accuracy. Weaknesses and limitations come into full play for the proposed installations. Shifting of the IPI arrays and resulting false readings may be generated due to construction vibrations. I suggest that as a minimum parallel casing (possibly in the same borings) be installed to verify IPI results with the manual system when the automated results are in question.

1-0 Evaluation **Concurred**
Please see response to Comment 3441042 above. The vibrating wire IPI sensors have a damping reservoir that can be filled, in the field, with a viscous fluid if unstable readings due to vibration are observed.

1-1 Backcheck Recommendation **Open Comment**
Recent experience has shown that the mechanical array arrangement of casing grooves, sensors, rods and joints can shift due to vibration, even in water-filled casing. False readings were proved out by extracting the IPIs, doing manual surveys and reinstalling the IPIs after reinforcing the joints and modifying the suspension. Additional inclinometers were also installed so regular manual surveys could be continued as a check. Please comment back.

2-0 Evaluation **Concurred**
Inclinometer casing will be added for the previous piezometer only installations near the IPI instruments as suggested please see response to

Comment #3441042.

2-1 Backcheck Recommendation **Close Comment**
I concur.

IEPR Comment 3441047	Other	Addendum H Table 3-2	Survey Points	n/a

The accuracy of survey points are of questionable value. Sight distances are over thousands of feet and accuracy may not be better than +/- 0.25 to 0.5 inches. Early trends from construction could be difficult to evaluate due to inaccuracies. Installation of several deep benchmarks (not called for) would better that accuracy, but it seems like multi-position borehole extensometers (MPBXs) have been over-looked in this context. MPBXs can conveniently be automated like piezometers. Accuracy of 0.01 inches or better could be expected with a range of 4 inches or more. It is more costly, but it would be balanced against saving on surveying labor in the long term.

1-0 Evaluation **Concurred**
We concur with the reviewer comment that making the survey monuments deeper would likely minimize impacts of near surface settlement due to heavy construction traffic. For the purposes of the DDR we have changed the dimensions of the survey monument to a 6-inch diameter by 10 ft long concrete post (original depth was 4 ft). Regarding the use of multi-position borehole extensometers (MPBX), if during the preparation of detailed design drawings and specifications the level of accuracy provided by MPBXs is required then we concur with their use and note that they can be readily automated with the proposed automated data acquisition system.

1-1 Backcheck Recommendation **Open Comment**
It is not so much the stability of the monuments that I question, it is the impact on accuracy that long sight-distances have. Deep benchmarks would be founded in the rock foundations at intermediate intervals to shorten sight distances from known elevations, hence securing better accuracy. Please comment back.

2-0 Evaluation **Concurred**
It is our opinion that the accuracy of the current planned survey monuments should be sufficient for the needs of this project. However, it is advised that during the final design stage when threshold values are determined, that the need for higher accuracy measurements be evaluated.

2-1 Backcheck Recommendation **Close Comment**
The leveling data will show the adequacy of the results. Deep benchmarks can be added later as indicated to improve accuracy if needed.

IEPR Comment 3441050	Other	Addendum H	3-17	n/a

Enclosures for data loggers and associated electrical components should all be rated NEMA 4X.

1-0 Evaluation **Concurred**
At a minimum, all installations will have something equivalent to a NEMA 4x enclosure within the installation. (Some items will have a NEMA 4x enclosure inside an outer enclosure which will be rated NEMA 3R.)

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3441055	Other	Addendum H Drawing I113	Vertical Flow Meter Detail	n/a

The drawing is fine, but a note should added regarding need for calibration with this application. It was noted during the observation of trial installations during our field visit that these instruments are mostly idle. I suggest that the device be tested in a 12-inch pipe under "laboratory conditions" and calibrated for at least low flow rates.

1-0 Evaluation **Concurred**
We concur that some method of calibration is required. The technical specifications should address all calibration issues for all instruments including the vertical flow meters.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3441059	Other	Addendum H Drawing I113	Piezometer Installation	n/a

The bottom detail is confusing in showing a slotted screen detail at the bottom of the riser pipe. It infers the installation of an open standpipe piezometer as well as 2 VWPs. I think it was meant to show the tremie pipe which does not have a slotted screen and is not termed a riser.

1-0 Evaluation **Concurred**
The detail was modified to remove the hatching implying a slotted screen and the label for riser pipe was deleted.

1-1 Backcheck Recommendation **Close Comment**

IEP Comment 3441060	Other	Addendum H Drawing I113	Subsurface Displacement Monuments ID	n/a

There are two such tables on the drawing with identical labels, but different coordinate values.

1-0 Evaluation **Concurred**

Drawing I101 has been updated to delete the incorrect table.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3441062	Other	Addendum H Drawing I 112	n/a	n/a

Piezometer and inclinometer installations are shown at 10-foot offsets on both sides of the wall. Concrete panel walls are most often constructed inside a pair of footings (guide walls), also serving as machine foundations. Make sure installations are well outside the guide walls and outside the action radius of the machine. The instruments do not have to be so close to the wall in my opinion. Offsets of 30-40 feet would be OK

1-0 Evaluation **Concurred**

The grouted piezometers on the upstream side of the seepage barrier are designed such that the sensor cables will be buried 24 inches below ground surface and will then run laterally to the RIO unit which is located up to 50 ft from the barrier wall. For the IPI and grouted in-place piezometer locations on the downstream side of the barrier wall we concur that 10 ft may be too close and may interfere with construction equipment even though it is located on the upstream slope. We have added a note to Drawing I-112 to indicate that the final location of the IPI/PZs will be approved by the Contracting officer once the contractor's means and methods are known.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3441064	Other	Addendum H Drawing I 112	Piezometers	n/a

The piezometer sensor layout has two staggered arrays of 3. I would make them two symmetrical arrays of 4 from 50 feet above the bottom of the wall to 25 feet below the wall.

1-0 Evaluation **Concurred**

One of the arrays has been modified to include a fourth piezometer. The two arrays will each have 3 piezometers symmetric from 50 feet above the base of the wall to the base of the wall. The damside array will have an additional piezometer 25 feet below the wall.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3441070	Other	Addendum H Drawing I 101	List of Piezometers	n/a

There is no list of retrofitted open standpipe piezometers for automation with VWPs. and no details on the following drawings for how to retrofit. To make such retrofits more effective for measuring artesian conditions, staying frost free and respond better the sensors should be sealed off below the lowest water level. A special packer, mechanical or

hydraulic may be considered for this.

1-0 Evaluation **Concurred**

There are a total of 4 existing piezometers which are to be automated. For measurement of artesian conditions, it was anticipated that the 24 pairs of new piezometers being installed would provide sufficient data to observe the artesian conditions. (All four of the existing piezometers are within 120 feet of a new piezometer. All new piezometers will be grouted in place and will provide data during artesian conditions.)

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3441075	Other	Addendum H Drawing I 100	Piezometers	n/a

It is not clear that existing piezometers marked on the plan are to be automated., only D-06-29. There are no RMU or RIO plotted near all existing piezometer plotted. If there are to be some hard wiring to the RIOs it should be drawn in on the plans.

1-0 Evaluation **Concurred**

The detail on Drawing I113 was updated to state the name of the four piezometers to be automated. The location for cabling is not shown for clarity. We added a reference on Drawing I113 for the RIO or RMU that it is intended to read the individual piezometers.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3441082	Other	Addendum H Drawings BI-102 thru BI-115	Piezometers	n/a

RMUs and RIOs are not marked on these drawings. It should also include any hard wiring paths where applicable. Hard wiring in near surface trenches should be surge protected with a heavy gage copper wire installed 6 inches above cables in the trench. Alternatively, steel conduits could be used for the cabeling.

1-0 Evaluation **Concurred**

RMUs, RIOs, and hard wiring locations were not marked on these drawings for clarity. We added a note to the drawings to state this. The RMU and RIO locations are shown on Drawing I100. The design typically uses either vibrating wire (VW) or 4-20 ma sensors. The VW sensors have a surge protection module built into the sensor housing. Any cable over 50 feet in length uses an external surge protection module at the surface before the horizontal run. Additionally, at the input of the multiplexer located at the datalogger device, there is also surge protection. All surge protection modules will be properly grounded to a ground rod located within 6 feet of the module. 4-20 ma sensors also go through the same type of surge protection modules at each end of the wire. In every case, the surge protection module is oriented to minimize damage to either the sensor at one end or the electronics at the other end of the cable.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 3450797	Hydraulics	Section 3, and Appendix A, Section 7	Pg. 8-9	n/a

The HMR52 model data was provided by the COE containing various watersheds in the study reach. Bolivar watershed was isolated and HMR52 was run as a single basin. A. From HMR 51, the preferred storm orientation was verified to be 230 degrees. The optimized storm orientation from HMR52 was computed to be 306 degrees. The difference in selected storm orientation does not affect the results significantly. B. The computed 6-hour precipitation totals ordered based on their positions during the storm event are given below. These values are identical to those given by the COE is given in Appendix A, Table 2. The order of 6-hr increments based on precipitation amounts are: 12-10-8-6-4-2-1-3-5-7-9-11 (where 1 is the largest 6-hr precipitation, 12 is the smallest 6-hour precipitation). C. There are other storm distributions accepted by HMR52. These storm distributions place the largest 4 maximum 6-hour precipitation periods towards the end of the event (instead of centering). One such arrangement is given as: 12-11- 10-9-7-6-5-3-1-2-4-8. As a result of a skewed distribution, the peak precipitation occurs after the ground is saturated and infiltration losses are at a minimum. Previous major events listed in Appendix A confirm more severe conditions under saturated soil conditions. COE takes this process into account in their simulations by applying a 30% event 8 days prior to PMF event. However, the precipitation losses listed in Appendix A, Table 2 show uniform distribution of 0.3 in/6-hr, and do not reflect saturated ground conditions properly. D. Model runs should be repeated with skewed storm distributions for sensitivity analysis. E. The computed peak PMF discharge can be affected by 10-15%.

1-0 Evaluation **Concurred**

The rainfall can be oriented to produce a higher peak discharge on the synthetic hydrographs. This would also potentially produce a higher peak pools in the reservoir. The rainfall is symmetrically applied to the unit graph based on the lag between the 3-5 day dry spell used for the storm. These discharges are placed in the HEC-Reservoir simulation program to determine the flow hydrographs to be used in the HEC-RAS model. This process takes several days and to recalculate the flows would be a significant task. If this task were completed, the whole process of generating the profiles would have to be redone and delivered to Planning if it produced any significant differences. The different frequencies were developed by taking percentages of the PMF and running the Reservoir Simulation Program to produce the requested pool elevation. These pools were reproduced fairly accurately and the process probably would not warrant the extra time and money to evaluate.

1-1 Backcheck Recommendation **Close Comment**

The response is adequate. Considering that the analysis uses a 30%PMF antecedent event prior to applying the PMF conditions, the additional work in revising the entire analysis is not warranted.

IEPR Comment 3450799	Hydraulics	Section 2 and Appendix A	Page 5 and Throughout Appendix A	n/a

On Section 2.2.1, Page 5, it is stated that after Dam Safety Assurance program, in 1989 the top of the dam was raised from 982 feet to 985.5 by the construction of a 3.5-foot high parapet wall along the upstream face of the dam and the spillway was widened to address the hydrologic deficiency. All hydraulic and hydrologic calculations use the top elevation of the dam as 982 feet. If there was a need to raise the dam, why is the new elevation not being used in the analysis? The additional height would change the hydrologic and hydraulic computations. The surface area of the reservoir at Elevation 982 is approximately 11,000 acres. The raised elevation would introduce an additional 40,000 acre-feet of water into the system without altering the dam failure times significantly.

1-0 Evaluation **Concurred**

The parapet wall was constructed not for additional storage but to contain several variables associated with a storm of the magnitude of the PMF. One of the major variables would be wave runup. The parapet wall was not constructed to add additional storage to the project but only confinement of a large storm such as the PMF

1-1 Backcheck Recommendation **Close Comment**

Agree, the parapet wall's primary use should be for wave runup.

IEPR Comment 3450801	Hydraulics	2.2.3 and Appendix A	n/a	n/a

Section 2.2.3 of the DDR states that after the DSA Program in 1989, the spillway was widened to 540 feet. A concrete sill was placed at the crest, as well as 230 feet downstream of the crest. The downstream cutoff is 15 feet deep. The spillway design flood has a peak inflow of 196,000 cfs with a freeboard of 5.4ft. a) From the historical profiles of the spillway (1981) given on Page 1135 and the boring samples (1981) on Page 1144 of the Draft DDR dated 5/28/2010, the material along the spillway past the downstream sill is highly fractured (e.g. sample C-8) at elevations below the cutoff wall. Subjecting this material to high spillway velocities may result in severe erosion downstream from the concrete pad. Also, changing the material from concrete to natural materials concentrates the local scour downstream from the concrete pad. I recommend that this section of the spillway be reanalyzed for local scour. b) There are no freeboard calculations. From practical engineering point of view, 5.4 ft of freeboard for such large discharges is not adequate. c) Spillway operation at PMF event is critical for the entire structure.

1-0 Evaluation **Concurred**

A spillway erodibility analysis was performed as part of the Major Rehab studies for this project. The details of that analysis are provided in Appendix C of the MRR. The analysis used SITES software to model the total outflow through the spillway (to simulate the possibility of all gates being stuck in the closed position). Hydrograph flows from the spillway crest up to the PMF were modeled in SITES. The results of the analyses showed that even during a PMF event some erosion does occur in the spillway (as expected) but the weir does not fail and a breach does not occur. The USACE Risk Assessment cadre is also currently looking at spillway erodibility as a potential failure mode for Bolivar Dam. The system response curves for this failure mode will be available near the first of October. If the cadre determines that the probability of failure is high and the risk associated with this failure is not at a tolerable level, further studies will be performed. This includes taking another look at the freeboard calculations.

1-1 Backcheck Recommendation **Close Comment**
The response is adequate. The IEPR panel should be advised on the outcome of Risk Assessment cadre's findings regarding erosion and freeboard calculations.

IEPR Comment 3450806	Mechanical	Appendix E	Pg. 1182 of DDR, Pg. 6-8 of Appendix E	n/a

Probability of Failure Charts on pages 6-8 of Appendix E (page 1182 of the DDR dated 5/28/2010) refer to Mohawk Dam. If the system is identical to Mohawk Dam, the text should make that clarification. Otherwise, the charts pertaining to Bolivar Dam should be used. Change is editorial.

1-0 Evaluation **Concurred**
The Fault Tree was copied but the specific numbers were changed to represent Bolivar as the machinery arrangement is identical. This is a typo that will be corrected.

1-1 Backcheck Recommendation **Close Comment'**
The response is adequate.'

IEPR Comment 3450820	Hydraulics	Page 12 and throughout Appendix A	Pg. 1182 of DDR, Pg. 6-8 of Appendix E	n/a

On Page 12 of the DDR, it is stated that the breach plan for Bolivar used an initial piping elevation of 910ft, and a trigger failure water surface elevation of 961.6 ft for different storms. The results of piping failure computations were partially verified by using NWS s BREACH model. A) Using the National Weather Service's BREACH model with an initial reservoir elevation of 962ft and piping elevation of 910ft with a side slope of 0.5V:1H, I computed peak flows from piping failure (BOL910A.OUT). B) I also computed two additional cases with piping failure elevations of 910ft with side slope of 0H:1V (BOL910B.OUT) and piping failure elevation of 905ft with breach side slope of 0H:1V. C) In general terms, in BOL910A.OUT, the computed breach parameters of time of failure (0.40 hr), bottom width (131 ft) are within the bounds of the COE study. However, the BREACH model predicts larger peak outflows (398,157cfs for piping at elevation 910ft

with side slope of 0.5V:1H as opposed to the COE'S 296,400cfs peak outflow from HECRAS). D) In BOL910B.OUT, for vertical wall breach geometry the computed breach parameters show the time of failure to be 0.64hr and the breach bottom width to be 67ft. The resulting peak discharge is 211,800cfs. E) In BOL905.OUT where the piping elevation is assumed to be at elevation 905ft, for vertical wall failure geometry the computed breach parameters show the time of failure to be 0.33hr, the bottom width to be 72ft and the computed peak discharge is 258,172cfs. During the field trip to the site, at several locations at the toe region the piping elevation was shown to be 905ft or lower. Therefore this elevation must be included in the analysis. F) Modeling results show a discrepancy between NWS's BREACH model and the COE's HECRAS model. Even though some of these differences is due to approximated field conditions in BREACH modeling (size, porosity, and unit weight characteristics of the surface material on the embankment), it is believed that the main difference lies in the fact that the BREACH model computes breach geometry and time of failure whereas these parameters are input by the user in HECRAS. The sensitivity analysis should be conducted using the range of computed bottom width and time of failure parameters using the BREACH model results as well as variations in breach side slope angles.
(Attachment: _Hydraulics_Ref_Files.zip)

1-0 Evaluation **Concurred**
As stated the National Weather Surface model and the HEC-RAS model produce different results. The parameters used in the HEC-RAS model were the best estimates of failure slope, bottom width, and failure elevation provided by the Geotechnical people at the time the model was being set up. The District has attended classes taught by Danny Fread of the Weather Surface who developed the Dambreak model for the National Weather Surface. We currently have some of this old model that was completed back in the 1970's. Mr. Fread commented in the classes that the model had been rewritten that he recommended using the "Flood Wave" model instead of using the original Dambreak model. As stated in the comment, no sensitivity was completed in completing the Bolivar study, but there is no reason that several variables could not be evaluated and some determination be made as to whether the economics needed to be reevaluated. This would also need to be a team decision and the time and money would have to be available.

1-1 Backcheck Recommendation **Close Comment**
I have used NWS's BREACH model, which is a different model than the DAMBREAK model that USACE Evaluator refers to (yet another model by Dr. Fread). The BREACH model does not route flows through river systems like DAMBREAK, FLOODWAVE, or HEC-RAS models, but rather computes breach parameters and failure times to be used in hydrologic models (HEC-HMS, HEC-1, etc). Whatever the tools, I agree with the Evaluator that the sensitivity analysis is a team decision and at this point there is no need for additional analysis.

IEPR Comment 450823	Hydraulics	2.2.3 and Appendix A	n/a	n/a

Page 15 of the Appendix A lists the input data used in the simulations as: Breach Base Width=100ft; Breach Invert=905ft; Breach side slopes=0.5 to 1; Time to complete formation of breach=0.5 hr; Reservoir Inflow varies for different pool levels. A) Using the HEC-1 dam break option, with the reported PMP inflow values from the DDR and different breach failure times and geometries, series of 5 scenarios were tested. Results are given in (BOLIVAR1.OUT). B) Using Breach widths of 100ft, 150ft, 150ft, 150ft, 200ft and failure times of 0.5hr, 0.75hr, 1 hr, 0.5hr, and 0.5hr, the maximum outflows were computed as: 199,250cfs, 262,475cfs, 260,787cfs, 264,171cfs, 328,751cfs. The ranges of variation in width and failure times are consistent with HECRAS results reported in the COE study. C) Using an invert elevation of 905ft, for a breach width of 200ft and failure time of 0.5hr, the HEC1 results show a peak discharge of 328,751cfs. The peak discharge from HECRAS is 296,400cfs. D) The discrepancy between computations from HEC1 and HECRAS is due to piping failure elevation. The analysis should consider placing the bottom elevation of piping to Elevation 905ft
(Attachment: BOLIVAR1.OUT)

1-0 Evaluation **Concurred**
Any of these variables could be re-evaluated based on a TEAM decision along with time and money to complete. This could have a significant impact on the project. There would be point in redoing the hydrology and hydraulics if it were not evaluated economically. HEC-RAS has had extensive use in the evaluation of Dam Failures and I am sure that even if two different modelers set the model up, they would come up with slightly different answers. It is obvious that increasing the widths , slopes, and invert elevation will produce a larger volume of outflow. The PDT team will go over the breach parameters.

1-1 Backcheck Recommendation **Close Comment**
Response is adequate. Failure time of 0.5hr is conservative (greater than 2.5 ft of erosion per minute) and may offset the selection of other non-conservative parameters (breach side slope, width, etc.). The PDT team should review these parameters.

IEPR Comment 3450828	Hydraulics	5.1	14	n/a

A concrete seepage barrier is proposed between embankment stations 20+00 and 64+00. The barrier will be installed to an average depth of 145 ft along its embankment alignment. A gravel layer at elevation 930 extends between north abutment and station 53+00. It appears that using a wider and shallower seepage barrier would intercept the flows through the gravel layer. Various alternatives were examined to optimize costs and to derive optimum configuration. Was the FEM modeling with deeper/narrower configuration compared with the shallower/wider configuration for seepage forces at the toe region? This, ultimately should be the deciding factor since the entire effort is to improve the safety of the structure. The purpose of the seepage barrier is to minimize

sources of excessive seepage. The selection of barrier configuration should reflect sound engineering computations.

1-0 Evaluation **Concurred**

The analyses to date did consider a shallower configuration of the seepage barrier. The DDR text will be revised to demonstrate/summarize this work, and show how the factors of safety with the shallower barrier did not yield an overall adequate/optimized remedial scheme relevant to the governing design guidelines. How and why the current proposed barrier geometry is deemed adequate/optimized will be further elaborated on in the revised DDR. The width of the barrier in the responder s opinion is not an overly sensitive parameter in terms of the modeling results (i.e. from numerical standpoint doesn't mean as much as practical considerations if looking at 2 versus 3 feet thick), however, the actual width of the barrier will be further considered during plans/specifications/cost analyses work. At a minimum existing state of practice will be followed, which for this type of barrier likely will be 3 foot wide excavation to ensure 2 foot thickness along vertical extent of barrier.

1-1 Backcheck Recommendation **Close Comment'**

Response is adequate.'

IEPR Comment 3450830	Hydraulics	2.2.3 and Appendix A	14	n/a

Page 15 of the Appendix A lists the input data used in the simulations as: Breach Base Width=100ft; Breach Invert=905ft; Breach side slopes=0.5 to 1; Time to complete formation of breach=0.5 hr; Reservoir Inflow varies for different pool levels. None of the cases in the main DDR report or the Appendix A consider failure due to overtopping. Since there is a 3.5ft wall added to the main embankment of the dam, and since there are doubts about the age of the outlet gates, it is possible to overtop the embankment. Especially, if a 30%PMF event (as it is considered in the study) is assumed to occur as antecedent storm, the reservoir elevations will be high enough to overtop the embankment. My initial calculations using the NWS BREACH model show the discharge from such events to be over 500,000cfs. The embankment overtopping should be considered with less severe antecedent conditions using overtopping elevations of 482ft and 485.5ft (with and without parapet wall failure).

1-0 Evaluation **Concurred**

Again, any of the variables could be changed. As far as the models go the District has used the HEC-RAS model to evaluate the failure. The guidelines are spelled out as to what percentages are acceptable for an antecedent event and how much time is reasonable between the antecedent and the main event of the storm. Again the parapet wall was not designed to accommodate additional storage. The District was not task to compare answers arrived at by using different models but this might be a topic of discussion. Which model more accurately represents what will happen when the Dam Fails. Danny Fread was an outstanding leader in the field of estimating damages created by a Dam Failure with his model.

1-1 Backcheck Recommendation **Open Comment**

This comment is directed on the potential mode of failure, rather than computational methodology. The mode of failure in simulations is the piping failure. The question the comment poses is: "was the main embankment overtopping simulated?" If the embankment height was established to avoid overtopping, the answer would be "yes." However, using a 30%PMF event as antecedent condition places an additional constraint on computations and may result in dam overtopping (regardless of the model used in the analysis, HEC-1, HEC-HMS, RES-SIM, etc).

2-0 Evaluation **Non-concurred**

The height of the 3.5 parapet wall (top elevation of 985.5 feet) was designed using the old DSA criteria to allow the project to pass the antecedent storm and PMF with the required freeboard (5.4 ft.). This design assumed 1 gate in each tunnel unoperable and full uncontrolled spillway flow (approximately 17,700 cfs through the two sluice gate tunnels and and 116,000 cfs through the uncontrolled spillway at elevation 980.1). At elevation 981.8, uncontrolled spillway flow will increase an additional 17,700 cfs above the 980.1 flow. Based on these figures, even if all 6 sluice gates stuck shut in the closed position, the uncontrolled spillway can pass enough flow to compensate for the sluice failure while still maintaining 3.7 foot of freeboard on the parapet wall during the PMF. For this reason and that it is expected that the dam will fail due to piping well before these levels are attained, an overtopping mode of failure was not considered.

2-1 Backcheck Recommendation **Close Comment**

I agree with USACE response. The hydraulic design was based on enough conservatism that for the given inflow hydrology, overtopping the dam is highly unlikely. Considering that overtopping cannot take place even if all 6 gates are stuck in closed position, then conducting an overtopping analysis is not needed. I agree with USACE that at high heads, the dam would most likely fail due to piping before overtopping conditions are encountered. However, the designers must recognize that 3.7 foot of freeboard during extreme discharges is not adequate and must be reconsidered. Once the dam is rehabilitated to avoid piping failures, it should still function to pass the PMF safely.

APPENDIX C'

SERVICE GATES REPLACEMENT 100% P&S IEPR

Comment ID	Discipline	DocType	Spec	Sheet	Detail
IEPR Comment 4595452	Hydraulics	Plans and Specs	n/a'	Construction Plans, Service Gates Replacement, Page 35, Detail C and Detail F	n/a

In these details, the upstream nose region of the gate is shown to be rounded. This is not an issue when the gate is fully open or closed. However, the Specifications for BIG SANDY CREEK OF TUSCARAWAS RIVER, BOLIVAR, OH, BOLIVAR DAM, SERVICE GATES REPLACEMENT, Appendix A – Climate and Hydrology Data, Pages 445-476 show that the tailwater levels submerge the gates partially or fully during the normal operation of the gates for many days. Under these conditions, the rounded-nose entrance type causes cavitation, vibration, and down pull. The rounded-nose design was used in the 30's and 40's and commonly experiences these undesirable effects. The designers must explore other gate entrance types.

1-0 Evaluation **For Information Only**

In general, the current Service Gate knife edge has performed satisfactorily for the last 74 years. The suggested updates in the "upstream nose region of the gate" will lead to significant cost increase due to the necessary changes in the existing embedded metals, masonry and adjustable water seal. Due to the satisfactory performance and significant cost increase of the proposed changes, the designer has only updated the Service Gate frame to welded construction but maintained the existing gate features (including the knife edge, sealing and track system) compatible with the existing recesses and machinery.

1-1 Backcheck Recommendation **Close Comment**

Agreed. The proposed changes were to improve performance of the gates for cavitation, vibration, and down-pull. If performance acceptable, no need to make changes.

IEPR Comment 4595458	Hydraulics	Plans and Specs	n/a'	Construction Plans, Service Gates Replacement , page 37, Section B-B	n/a

In these details, an indentation on the floor is shown for providing a water-tight bottom sealing of the gate when the gate is in closed position. This indentation would cause undesirable cavitation and would collect debris. The floor should be smooth and not have a

groove. With the development of new materials, a seal mounted at the bottom of the gate can provide adequate water-tightness when compressed at the closed-gate position. This type of bottom seal is shown on Page 34 of the current design and is adequate.

1-0 Evaluation **Concurred**

There is currently no indentation on the sluice invert (El. 895.0) On the other hand, the existing Adjustable Seal, is durable, performs well and is compatible with the current design.

1-1 Backcheck Recommendation **Close Comment**

Agreed. The proposed changes were to improve performance of the gates. If performance acceptable, no need to make changes.

IEPR Comment 4595464	Hydraulics	Plans and Specs	n/a'	Construction Plans, Service Gates Replacement, page 25, Detail F	n/a

In this detail, an indentation on the plan view behind the gate is shown. This is to accommodate room for the gate frame. This indentation in the plan view could cause undesirable cavitation and would collect debris. The gate frame can be modified/lifted 6"-12" from the floor and the square indentation can be eliminated by using a gradual angular contraction along the diagonal line.

1-0 Evaluation **Concurred**

The indentation on the lower portion of the existing Service Gate Frame recesses could cause undesirable cavitation and would collect debris. However the existing Service Gate and recess design is 74 years old and has generally performed well. The proposed gate frame and recesses updates would alleviate the debris and cavitation issues but would increase the scope and cost of this work significantly. The 6"-12" gate frame modification would complicate the gate frame fabrication, introduce significant changes to the existing embedded metals, steel liners and recess masonry.

1-1 Backcheck Recommendation **Close Comment**

Agreed. The proposed changes were to improve performance of the gates. If performance acceptable, no need to make changes for minor gains.

IEPR Comment 4595471	Hydraulics	Plans and Specs	n/a'	Specifications for Bolivar Dam Service Gates Replacement, pages 3 and 4	n/a

Add "Wire ropes for the hoist shall be of stainless steel" to text. The wire rope specification

is clarified later in Section 35 01 43, on page 364. However, in this earlier section, a sentence can be added to stress the point.

1-0 Evaluation **Non-concurred**
The wire ropes are not specified to be stainless steel as the ability to get the required strength will not be possible.

1-1 Backcheck Recommendation **Close Comment'**
Evaluator response adequate.'

IEPR Comment 4595479	Hydraulics	Plans and Specs	n/a'	Specifications for Bolivar Dam Service Gates Replacement, pages 3 and 4	n/a

Hydraulic cylinder operator is preferable instead of the rope drum type. The current design uses the original gate hoisting system. Hydraulic cylinder is an optional newer technology currently being used in gate installations.

1-0 Evaluation **Non-concurred**
Your suggestion would be a complete redesign and would not be considered a rehabiitation. Also, it would require a much higher cost than the rehabilitation being performed. Lastly given the operation of the gate, this would not be feasible as the gate drops onto the seals rather than merely sliding into position. A cylinder operated gate would require modification to the embedded seals. Multiple other issues would preclude the option of changing from a "winch hoist" to a "cylinder hoist".

1-1 Backcheck Recommendation **Close Comment**
The USACE Evaluator response is adequate.

IEPR Comment 4595485	Hydraulics	Plans and Specs	n/a'	Construction Plans for Bolivar Dam Service Gates Replacement, pages 1-65	n/a

No trash racks appear in drawings. Due to large amounts of wooded debris shown in the photos and observed during the site visit, the use of trash racks is recommended. Since Bolivar Dam is a dry dam, the blockage of the intake gates by trees may impede the optimal operation of the reservoir, but does not pose an immediate danger.

1-0 Evaluation **Non-concurred**
Project personnel has been able to manage the debris with current Service Gates operation procedure. The cost of fabrication, installation, operation and maintenance of trash racks did not appear to be cost effective. The collection and

disposal of the debris will add to the project operations and maintenance costs.

1-1 Backcheck Recommendation **Close Comment**
Agreed. The proposed thrash racks were to improve operation of the gates. If performance acceptable, no need to make changes.

IEPR Comment 4595496	Hydraulics	Plans and Specs	n/a'	Construction Plans for Bolivar Dam Service Gates Replacement, pages 33-35, ST 210-212	n/a

The Specifications for BIG SANDY CREEK OF TUSCARAWAS RIVER, BOLIVAR, OH, BOLIVAR DAM, SERVICE GATES REPLACEMENT, Appendix A – Climate and Hydrology Data, Pages 445-476 show that the tailwater levels submerge the gates partially or fully during the normal operation of the gates. Under these conditions, in addition to the upstream skin plate a downstream skin plate is desirable for sediment management of gates. The construction plans for Bolivar Dam Service Gates Replacement, pages 33-35 show a downstream plate but it is not clear if this is a skin plate or not. Sediment management is important for the fluid operation of gates but during extreme events, gates pass only a small fraction of discharge.

1-0 Evaluation **Non-concurred**
The current design does not require a downstream skin plate. However there is not a significant sedimentation problem with the current Service Gate layout. Addition of a downstream skin plate will lead to gate buoyancy issues and significant changes on the adjustable seal, gate geometry, bearing plates and center of gravity. Changes in the gate weight and center of gravity could lead to costly changes in the gate operating machinery. Furthermore the proposed change will increase gate weight, cost and the Government ability to inspect the gate frame.

1-1 Backcheck Recommendation **Close Comment**
Agreed. The proposed changes were to improve performance of the gates. If performance acceptable, no need to make changes.

APPENDIX D

SEEPAGE BARRIER 90% P&S IEPR

Comment ID	Discipline	DocType	Spec	Sheet	Detail
IEPR Comment 4595250	Geotechnical	Plans and Specs	02 23 10	n/a	n/a

Section 02 23 10; 1.3.2 – Site Conditions and Geology Data (p. 3). The paragraph states that site geology consists of horizontally bedded sedimentary bedrock consisting of carbonaceous shale and indurated clay. It also states that the strength and modulus of the bedrock vary widely, from deteriorated rock of very low strength to rock with compressive strength in excess of 30,000 psi. Is the deteriorated nature of the rock due to the presence of fractures? Is information about bedrock discontinuities (e.g. # of joint sets, orientations, continuity, aperture, etc) available? If so, it should be provided in the report because grout mixes and grout takes in fine-grained rocks like shale and claystone will depend upon the nature of discontinuities.

1-0 Evaluation **Concurred**

The deterioration of the bedrock is generally due to valley stress relief and weathering. The joints have become more open due to the valley stress relief and weathering has occurred due to exposure along the ridge. Detailed information regarding the discontinuities is provided in the core logs and the Optical Televiewer Imaging Report. This report is available in the DDR and will be presented in the Geotechnical Baseline Report as part of the Plans Specs.

1-1 Backcheck Recommendation **Close Comment**

It would be useful if the information about discontinuities could be summarized as the number joint set present, spacing between joints, continuity of joints, joint aperture, etc.

Comment ID	Discipline	DocType	Spec	Sheet	Detail
IEPR Comment 4595256	Geotechnical	Plans and Specs	02 23 10	n/a	n/a

Section 02 23 10; 1.4 – Submittals (Grouting Records, p. 5). "The grouting report shall include the hole identification ---------, and a plot of the evolving permeability and gauge pressure and time". Will permeability be determined during water pressure testing or during stage grouting operations, or both? What does the term "evolving" refer to?

1-0 Evaluation **Concurred**

"Evolving" refers to the change in permeability due to grouting operations. Pressure testing will be performed before and after grouting operations.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595317	Geotechnical	Plans and Specs	02 23 10	n/a	n/a

Section 02 23 10; 3.4 – Grout Hole drilling in Rock and Concrete (p. 9). "It is anticipated that the required depth of radial grout holes will not exceed 30 feet". What is the basis for the anticipated depth of 30 feet?

1-0 Evaluation **Concurred**

This was originally based on the layout of the seepage barrier and it was anticipated that the grout holes would not need to extend beyond 30-feet to effeciently tie-in to the seepage barrier; however, based on input from construction experts and design team reviews this remark will be removed and it may be necessary to have holes extend beyhond 30-feet.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595321	Geotechnical	Plans and Specs	02 23 10	n/a	n/a

Section 02 23 10; 3.4 and 3.5 – Radial Grout-Hole Drilling and Grouting (p. 9 and 10). On page 9, the report states that holes shall be drilled beginning at the crown and then proceeding toward the invert by drilling, in order, the next lower holes. On page 10, the report states that the bottom holes (at the invert) shall be grouted first, proceeding then to the crown by grouting the next higher holes alternately from one side to the other. Why are the sequences of drilling and grouting reversed? Also, does it imply that all radial holes will be drilled first before they are grouted?

1-0 Evaluation **Concurred**

This was an inadvertent discrepancy. The holes should be both drilled and then grouted beginning at the crown of the tunnel.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595330	Geotechnical	Plans and Specs	03 30 00	n/a	n/a

Section 03 30 00; 1.5 - General Requirements; 1.5.2.1 – Strength Requirements (p. 5). On page 6, under part-a of compressive strength testing, the report states that compressive strength will be measured on properly cured samples (ASTM C31/C31M) and will be considered satisfactory so long as the average of all sets of three consecutive test results equals or exceeds the specified compressive strength and no individual test result falls below the specified strength by more than 500 psi. On the same page, under part-b of compressive strength testing for in place concrete that is considered potentially deficient, the report states that strength will be measured on at least three representative core samples, taken from a location approved by the Contracting Officer, and tested in

accordance with the ASTM method C42/C42M. The concrete in the area represented by the cores will be considered satisfactory if the average strength of the cores is equal to at least 85 percent of the specified strength requirement and if no single value is less than 75 percent of the specified strength requirement. Why different specifications are being used for cured cylinders versus in-place concrete cores?

1-0 Evaluation **Concurred**

The tests requirements will be changed in section a. to state that compressive strength tests should not be less than 75% of specified strength (750psi).

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595336	Geotechnical	Plans and Specs	03 30 00	n/a	n/a

Section 03 30 00; 2.2 – Aggregates; 2.2.2 Aggregate Quality (p. 10). Subsection 2.2.2 provides specifications for the aggregate delivered to the mixer. It contains a Table that lists acceptable values for specific gravity, absorption, durability factor, organic impurities, L.A. abrasion loss, petrographic examination, and coal and lignite of specific gravity less than 2.00. Regarding petrographic examination, the footnote to the Table states that chert content of coarse aggregate shall not exceed 1.0 percent, including chert of any specific gravity, and 0.5 percent for concrete within 2 feet of all finished surfaces exposed to weathering. Research shows that it is not only the concrete containing chert but also the concrete containing shale particles and argillaceous limestone, with argillaceous material evenly distributed throughout the rock, that is susceptible to freeze-thaw damage in the form of popouts, pitting, and D-cracking. Petrographic examination should also look out for such deleterious materials. Additionally, alkali reactivity and insoluble residue test, which are mentioned in the later sections on concrete quality control, are not included in the Table.

1-0 Evaluation **Concurred**

Alkali Reactivity and Insoluble Residue will be added to the table. in regard to limits on Shale and argillaceous material, does the reviewer recommend relying on petrographic examination to determine the content of these materials? What limits are recommended?

1-1 Backcheck Recommendation **Close Comment**

In soluble residue test should take care of the argillaceous limestone. The insoluble residue content should nor exceed 20% (preferably much less). Shale particles are particularly subject to F-T damage in the form of popouts and pitting. ASTM specifications for deleterious materials can be used for the limits. My recommendation is that the total percentage of chert, shale particles, clay lumps, and friable particles should be less than 3% by weight.

IEPR Comment 4595338	Geotechnical	Plans and Specs	03 30 00	n/a	n/a

Section 03 30 00; 3.9 Testing and Inspection for Contractor Quality Control (p. 29); subsection 3.9.2.3 – Quality of Aggregates (p. 30). This subsection contains a Table which lists the frequency of aggregate testing program (3 months, 6 months, 12 months) regarding various properties. This Table includes alkali aggregate reactivity and material finer than 200 sieve in addition to the properties listed in the Table on page 10 (referred to in comment 6 above). It will be better if the same tests are listed in these two Tables.

1-0 Evaluation **Concurred**

These tables will be adjusted to list all tests in both.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595340	Geotechnical	Plans and Specs	03 37 29	n/a	n/a

Section 03 37 29; Subsection 2.1.2.2 – Aggregate Quality (for seepage barrier concrete) (p. 13). Subsection 2.1.2.2 contains a Table lists requirements for the seepage barrier aggregate, both fine and coarse. The Table lists 18 different properties many of which are not included in the two Tables referred to in comments 6 and 7, such as clay lumps and friable particles, fineness modulus, flat and elongated particles, argillaceous material, strained quartz, chert with chalcedony, dolomite insoluble residue, CaO:MgO ratio, maximum total deleterious materials. I recognize that the seepage barrier is the most important component of this contract, will use pozzolans, and will require the most rigorous testing protocol for eliminating poor quality aggregate, and the associated problems, but some of the requirements listed for seepage barrier aggregate, e.g., clay lumps and friable particles, flat and elongated particles, argillaceous material, insoluble residue, are equally valid for aggregate used for structural concrete and should be included in the corresponding requirements.

1-0 Evaluation **Concurred**

Agreed. These limits will also be placed on the structural concrete materials.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595344	Geotechnical	Plans and Specs	03 37 29	n/a	n/a

Section 03 37 29; Subsection 3.3.4 – Concrete Placement (p. 24). The first sentence of this subsection states: "Concrete shall be continuously placed in a manner that will not cause segregation or honeycombing of concrete aggregates nor dislodge the excavation sidewalls into the wall element excavation ". I do not understand the second part of the sentence starting with the word "nor". Also, how will the honeycombing, if it occurs, be detected?

The report states that a borehole camera will be used to examine the holes drilled into the already constructed sections of the seepage barrier but the camera will miss the honeycombing occurring between the holes. Will any other means to detect honeycombing be used? Accurate detection of honeycombed areas will be important because such areas will need to be repaired. Subsection 3.3.4.2 (Repair of Unacceptable Concrete) states: "Unacceptable zones of concrete such as honeycombed, segregated, or uncemented zones, voids, cold joints found within the core boring shall immediately be repaired or removed and replaced by appropriate means". Again, I am not sure how the lateral extent of such zones will be determined for repair or replacement purposes.

1-0 Evaluation **Concurred**

Honeycombing and segregation will be detected in core samples and potentially in down-hole camera images. Also, if directed by the Contracting Officer, the Contractor will perform hydraulic pressure testing, which will alert the government to any potential openings or defects in the wall. If segregation is detected, additional borings will be directed until the extent of segregation in determined.

1-1 Backcheck Recommendation **Close Comment**

If segregation and honeycombing are detected, hydraulic pressure testing before and after replacement of defective areas, in my opinion, will be a very appropriate way to ascertain if the problem is resolved.

IEPR Comment 4595348	Geotechnical	Plans and Specs	03 37 29	n/a	n/a

Section 03 37 29; Subsection 3.4.2.3 – Quality of Aggregates (p. 28). There is a Table in this subsection that does not include some of the properties listed in the Table in Section 2.1.2.2 (p. 16), such as fineness modulus, argillaceous material, strained quartz, chert, chert with chalcedony, dolomite, insoluble residue, CaO:MgO ratio, and total deleterious materials. Will these properties not be evaluated on routine basis?

1-0 Evaluation **Concurred**

It is expected that the qualities such as fineness modulus can be verified through the sieve analysis to determine material finer than 75um and various mineral contents will be determined through petrographic analysis. The insoluble residue will generally not change significanlty as long as the contractor is obtaining materials from the same formation.

1-1 Backcheck Recommendation **Close Comment**

My experience with limestone quarries shows that the nature of material can change significantly from one ledge to another ledge within the same formation. However, if the Contractor can demonstrate that the formation from which the aggregate is obtained is uniform in nature, the assumption that insoluble residue will not change will have a valid basis (this can be accomplished by testing samples from different levels within the formation).

IEPR Comment 4595352	Geotechnical	Plans and Specs	31 00 00	n/a	n/a

Section 31 00 00; Subsections 3.10 (Embankments) and 3.11 (Backfilling) (p.12). The specifications for embankments require embankment material to be compacted to at least 98 percent of laboratory maximum dry density as determined by ASTM method D698 but compaction water content is not specified. On the other hand, specifications for the backfill require the material to be compacted in accordance with ODOT standard specs, i.e. compact to at least 98 percent of laboratory maximum dry density at 1% below to 3% above the optimum moisture content as determined by ASTM method D698. It is not clear to me why moisture content is specified in case of backfill but not the embankment. In my opinion, both density and moisture content should be specified for all compaction operations.

1-0 Evaluation **Concurred**

The intent of paragraph 3.11 Backfilling is to cover any backfill of soils in areas on site which involve fine grained soils; for example, fine grained (residual) soils exist in the left abutment where utility and other work will be completed. Embankments (para. 3.10) on the other hand, will consist of only random fill (granular soils) obtained from the onsite borrow area. Laboratory testing of soil samples from two borings completed in the borrow area show 6.8 and 4.7% fines (non-plastic) for soil classifications of GP-GM and SP, respectively. Due to the granular free-draining nature of the borrow soils (and embankment shell materials) to be used for fill for the work platform, an acceptable moisture content range was not deemed necessary. However, in response to this and in-house review comments the requirement for in-place density of random fill will be revised to the following: compact to at least 95 percent of laboratory maximum dry density and within plus or minus 3 percent of the optimum moisture content as determined by ASTM D698.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595356	Geotechnical	Plans and Specs	31 56 00	n/a	n/a

Section 31 56 00; Subsection 1.3.3 – Abutment Rock (p. 5). This section states: "Of particular relevance to Seepage Barrier construction, the limestone units in the left abutment contain persistent and solutioned discontinuities which have historically discharged several hundred gallons per minute when a Bolivar pool is retained, and unconfined compressive strength ranging from 5,000 to over 30,000 pounds per square inch". I have two questions regarding this statement: (i) will the information about the orientations and other characteristics of these discontinuities, particularly the apertures, be available to the contractor; and (ii) will these discontinuities be grouted before seepage barrier construction or will the seepage barrier be the only protection against water flow through them?

1-0 Evaluation **Concurred**

The orientation and spacing of the joints and bedding planes, as well as the conditions of these features (clay filling, smoothness, aperture, etc) in the bedrock are currently availble in the DDR and will be provided to prospective contractors in the Geotechnical Baseline Report. These discontinuities will not be grouted prior to construction of the seepage barrier. This decision was made due to the size of the defects and solution features in the rock and based on the fact that Bolivar is a dry dam.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595362	Geotechnical	Plans and Specs	31 56 00	n/a	n/a

Section 31 56 00; Subsection 1.9.2 – Quality (p. 11). For the test section of the seepage barrier, and for the entire length and depth of the barrier, the in-situ permeability needs to be 1x10^-6 cm/sec. How will it be ensured that the in-situ permeability measured from boreholes is representative of the entire seepage barrier?

1-0 Evaluation **For Information Only**

The true check of the effectiveness (effective permeability) of the barrier in reducing hydraulic heads is through monitoring piezometers upstream and downstream of the barrier during high water events and comparing to the seepage modeling results performed during the design phase (reference Bolivar Dam Major Rehabilitation Report and Bolivar Dam Design Documentation Report). Bolivar Dam is operated as a dry dam and therefore does not normally maintain a pool. Therefore, a significant pool needed to truly test the barrier s effectiveness is not within the control of the designer. The next best option to verify the barrier permeability is to perform permeability tests in the constructed barrier. Obviously we cannot drill holes every other foot along the barrier alignment for verification. However, verification holes will be drilled at numerous locations along the barrier alignment (not to exceed 50 LF spacing in demonstration reach and 200 LF for remainder of barrier) and these hole locations will be selected by the Contracting Officer. In addition, the specifications has been revised to include angled verification holes as well. Also, daily "bulk samples shall be taken from within the trench location soon after placement, or otherwise appropriate for the method as described in the Seepage Barrier Construction Plan, and used for quality control testing" and "additional samples of backfill material shall be taken and submitted for Government quality assurance testing as directed by the Contracting Officer." This is considered a suitable verification process and is a similar to the approach for confirming proper embankment construction by performing in-place density check tests at various locations in an embankment after each lift is completed. The specifications state that should verification testing reveal a failure to meet the acceptance requirements "the section of the Seepage Barrier between the locations of the two nearest passing tests shall be removed and replaced at no additional expense to the Government...." but "the Contractor may perform additional drilling and testing as specified herein at no additional cost to the Government to reduce the extent of segment to be replaced, when approved by the Contracting Officer."

1-1 Backcheck Recommendation **Close Comment'**
Thank you.'

IEPR Comment 4595367	Geotechnical	Plans and Specs	31 56 00	n/a	n/a

Section 31 56 00; Subsection 2.1.1 – Water (p. 11 12). This section suggests Sandy Creek and existing relief wells as the two possible locations for seepage barrier backfill mix. If water from relief wells is used, caution will have to be exercised that it is not pumped out at a rate faster than the inflow to prevent the potential for piping.

1-0 Evaluation **For Information Only**
Eight of these wells (W-21, 22, 23, 23A, 29, 31, 32, and 33) were pump tested in 2009 during rehabilitation work and showed only 2.8 feet average drawdown while pumping at 1000 GPM. Drawdown at 1000 GPM ranged from a minimum of 0.61 ft (W-21) to maximum of 5.91 ft (W-31). The depth to the water surface from top of riser in relief well W-31 is typically around 8 feet for lake elevations corresponding to 895-900 feet. The total depth of W-31 is 75 feet which leaves more than adequate buffer for drawdown from pumping efforts.

1-1 Backcheck Recommendation **Close Comment**
Thank you.

IEPR Comment 4595373	Geotechnical	Plans and Specs	31 56 00	n/a	n/a

Section 31 56 00; Subsection 3.7 – Seepage Barrier Measurements (p. 15). This section states that the contractor will identify the equipment "suitable to measure the depth of the seepage barrier at intervals no longer than 10 feet". Is "10 feet" the lateral spacing between these measurements or the vertical intervals at a given location? The subsection also states that the seepage barrier width shall be determined every 50 feet along the trench centerline and at locations directed by the Contracting Officer. The measurements will be taken in 10 foot vertical increments for the entire trench depth. From the above, I am not sure if the depth and width measurements will be taken at the same locations or not.

1-0 Evaluation **For Information Only**
For clarity, the first sentence of paragraph 3.7.1 will be revised to state that the following: "The Contractor shall identify in the Seepage Barrier Construction Plan procedures and equipment suitable to measure the depth of the Seepage Barrier at intervals no longer than 10 feet measured horizontally along the barrier alignment." The depth and width measurements will be taken at the same locations with more depth measurements taken than width measurements. For

example, a depth and width measurement will be taken at least every 50 feet, but depth measurements will also be taken at least every 10 feet within that 50 foot reach. The text will be revised to clarify.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595375	Geotechnical	Plans and Specs	31 56 00	n/a	n/a

Section 31 56 00; Subsection 3.7.3 – Verticality (p. 15). The subsection states: "The contractor shall ensure that the seepage barrier is constructed vertically". Is this requirement to be met by having a vertical borehole drilled from barrier centerline fall within the middle-third of the base width? In other words, what will be the tolerance limits for verticality?

1-0 Evaluation **For Information Only**

A barrier verticality tolerance was not directly stated in the specifications as the verticality of the barrier is not as critical as the continuity of the barrier. The barrier shall be vertical to ensure continuity, but deviations in verticality are not problematic as long the alignment transitions from these deviations in such a way so that continuity (2 ft. wide for entire depth along barrier length) is maintained. As noted in paragraph 3.9 part e., the "seepage barrier verticality shall be demonstrated through cores (by core containing backfill material only) taken in accordance with Section 31 57 00 VERIFICATION DRILLING AND TESTING." The specifications have been revised to also include angled verification borings. The requirement for adequate completion of these verification borings will indirectly control the necessary verticality tolerance of the barrier.

1-1 Backcheck Recommendation **Close Comment**

Thank you for the explanation.

IEPR Comment 4595378	Geotechnical	Plans and Specs	31 57 00	n/a	n/a

Section 31 57 00; Subsection 1.2.2 – Field Permeability Testing (p. 4). The section states that "Field permeability testing is the process of injecting water into the seepage barrier through pre-drilled holes for determining the in-situ permeability of the seepage barrier". What will be the spacing between the holes used for permeability testing (200 feet?)? Wide spacing may not yield data representative of the barrier sections between the holes.

1-0 Evaluation **For Information Only**

As discussed in paragraph 1.4, the 200 feet is a maximum spacing and "drill holes may be added or deleted as required by the Contractor Officer." Also, a maximum spacing of 50 feet is required for the demonstration section reach. In addition, per in-house review comments the specification has been revised to include a number of angled verification borings. This is deemed adequate coverage and spacing for barrier verification and limits excessive drilling and potential damage to the completed barrier.

1-1 Backcheck Recommendation **Close Comment'**
Angled borings is a very good idea.'

IEPR Comment 4595380	Geotechnical	Plans and Specs	31 57 00	n/a	n/a

Section 31 57 00; Subsection 1.4 – Sequencing and Scheduling (p. 5). The subsection states that: "The location of the verification drill holes shall be determined by the Contracting Officer, with a maximum spacing of approximately 200 feet along the centerline of the seepage barrier from Station 23+00 to its termination in the left abutment and a maximum spacing of 50 feet within the reach of the Demonstration Section from Station 20+00 to 23+00. Am I correct in assuming that the Demonstration Section, with 50-foot hole spacing, would have been tested prior to construction of the remaining barrier? Also, will the verification drill-hole testing be performed concurrently as the seepage barrier construction progresses?

1-0 Evaluation **For Information Only**
Yes, tested and approved - see third sentence of paragraph 1.9.2 Quality of Section 31 56 00 for additional discussion. Yes, the verification drill-hole testing will be performed concurrently as the seepage barrier construction progresses.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595389	Geotechnical	Plans and Specs	31 57 00	n/a	n/a

Section 31 57 00; Subsection 3.2.6 – Selection of Test Samples (p. 8). The sampling procedure states that five samples of core per boring will be selected for unconfined compressive strength. One sample will be taken from each of the following depth ranges: 10-25 feet (top zone), 25-50 feet (high zone), and 50-75 feet (middle zone); between 50 and 10 feet from the bottom of the seepage barrier (low zone), and within 10 feet of the bottom of the seepage barrier (bottom zone). The Contractor may be advised to take samples from different locations within these depth intervals, from one hole to another, so as to provide more information about the homogeneity of the seepage barrier.

1-0 Evaluation **Concurred**
The text states that "if the core includes zones of backfill of varying quality, samples shall be taken from each zone" and "additional samples may be required by the Contracting Officer to capture variability." The sentence stating "test samples shall be taken from depths as indicated above..." will be revised to "test samples shall be taken from locations as indicated above."

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595390	Geotechnical	Plans and Specs	31 57 00	n/a	n/a

Section 31 57 00; Subsection 3.4 – Field Permeability Testing (p. 9). This subsection states that: "The field permeability test shall be performed by filling the core hole with potable water. Water shall be added to a drop pipe to fully saturate the test zone". How wide will be the test zone and how will it be confirmed that it is fully saturated?

1-0 Evaluation **For Information Only**

The test zone will be the entire core hole as defined by paragraph 1.2.1 Core Drilling which states the core hole shall be of "required diameter over the entire depth (minus the bottom 2 feet) of the constructed barrier wall." Also, as per paragraph 3.3.1, "video recording shall be performed in a core hole full of water the same day as the field permeability test is performed and after completion of core hole cleaning."

1-1 Backcheck Recommendation **Open Comment**

I understand that the test zone will extend to the entire depth of the core hole. However, my question pertained to the lateral extent of the test zone. Isn't it the barrier material surrounding the core hole that comprises the test zone and that will be saturated?

2-0 Evaluation **For Information Only**

The sentence stating "fully saturating the test zone" is redundant and will be deleted. Note that the specs. state that "pre-soaking of the corehole is allowed."

2-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595396	Geotechnical	Plans and Specs	n/a	n/a	n/a

Proposal: Section 2.3, Part B – Seepage Barrier-Quality Control and Verification (Tab 1B), Subsection 2.3.1. This subsection of the proposal states: "Describe the quality control program for proving and documenting that the minimum dimensions, continuity, homogeneity, and quality requirements of the seepage barrier have been achieved". This statement does not include strength and permeability requirements which are important components of quality control. A homogeneous and continuous barrier of the required dimension may or may not meet the permeability requirements. Since measuring in-situ permeability in a manner so that it is representative of the entire length and depth of the barrier is a very important aspect of the quality control, I suggest including "permeability' in the quality requirements.

1-0 Evaluation **Concurred**

Will revise and include as requested.

1-1 Backcheck Recommendation **Close Comment**

4595411	Geotechnical	Plans and Specs	n/a'	Plans, page CG103	n/a

The seepage barrier is required to be installed a minimum of 10 feet in depth into bedrock from station 58+70B to approximately station 65+40B. Is the Contractor required to monitor seepage barrier installation with a geotechnical engineer to confirm the bedrock elevation and modify the seepage barrier installation depth where bedrock profile varies?

1-0 Evaluation **Concurred**

The Government, as well as the Contractor, will have construction inspectors on-site to determine location of top of sound rock and the final depth of the seepage barrier in these locations based on the top of sound rock.

1-1 Backcheck Recommendation **Close Comment**

I concur and this may be closed with no additional comment.

IEPR Comment 4595412	Geotechnical	Plans and Specs	n/a'	Plans, page CG101 to CG103	n/a

The seepage barrier from station 20+00B to 58+70B is to be installed to elevation 815.00. The soil types shown on Sheets BG703 to BG706 indicate a "Fine to coarse grained silty sand with intermittent gravel zones (SM-SP)." What analysis has been used to confirm the depth of the seepage barrier is sufficient to prevent seepage issues beneath this level and any potential piping? Has a flow net evaluation been conducted? Since gravel zones exist what consideration has been given to potential highly permeable zones beneath the proposed seepage barrier and bedrock? What is the potential risk and how was the el. 815 established as controlling?

1-0 Evaluation **For Information Only**

Questions regarding the lateral extent and depth of the proposed seepage barrier have already been addressed in a prior review (IEPR) of the Bolivar Dam Design Documentation Report – see attached comments and responses. Note that the risk assessment referenced in the comments is still ongoing but nearly completion. (Attachment: attachment_for_4595412.pdf)

1-1 Backcheck Recommendation **Open Comment**

Thank you for attachment 4595412.pdf discussing the background on full-depth seepage barrier. As I understand this document there has been concern over whether a full-depth seepage barrier should be used and you have indicated a risk assessment is currently being completed. I have read the document and still have concern over whether a full-depth seepage barrier should be considered. Your note indicates a risk assessment is being completed I would recommend leaving this comment open and re-assessing after the risk assessment is complete since it is a critical issue.

1-2 Backcheck Recommendation **Close Comment**

A teleconference was held with USACE personnel on June 13, 2012 to discuss Comment 4595412. Erich Guy with the USACE indicated the Illinoian geologic sequence is approximately from elevation 900 to bedrock and that it

is felt this sequence is not a susceptible to seepage with a seepage barrier extending part way into this sequence (currently set to extend to a bottom elevation of 815.00 from 20+00 to 58+70). The interpretation was that the seepage barrier would interrupt potential gravel layers and not provide a continuous flow path. Although I still have concern with a partial seepage barrier it is my understanding from the teleconference that the USACE is conducting a separate risk management assessment study and the assessment of the seepage barrier penetration depth will be reviewed prior to the start of construction in late 2012. It is recommended that additional assessment be completed prior to construction to assess risk based on using a partial seepage barrier. This item may be closed out with the understanding it will be addressed prior to start of construction.

IEPR Comment 4595414	Geotechnical	Plans and Specs	n/a'	Plans, page CG113	n/a

The existing note states "Maintain 5' min. clearance from top of existing tunnels. Recommend you consider adding to this note the following: "See profile for station limits."

1-0 Evaluation **Concurred**
The note will be added.

1-1 Backcheck Recommendation **Close Comment**
I concur and this may be closed with no additional comment.

IEPR Comment 4595417	Geotechnical	Plans and Specs	n/a'	Plans, page BG707 and CS701	n/a

Sheet BG707 depicts the bottom of seepage barrier at El. 926.65 while sheet CS701 depicts it at 916.65. Please verify whether this drawing needs to be modified to match the design drawings or verify why there is a difference between the two drawings. Drawing BG707 also shows the tunnel intercepting the seepage barrier.

1-0 Evaluation **Concurred**
Sheet BG707 is shown incorrectly. An alignment change was made on the design drawings that didn't get carried over to BG707. This drawing will be modified with correct alignment and elevation as shown on CG103 and CS701.

1-1 Backcheck Recommendation **Close Comment**
Thanks for checking this. This item may be closed without further comment since it is being addressed.

IEPR Comment 4595418	Geotechnical	Plans and Specs	n/a'	Plans, page CS701	n/a

This is the only drawing I found that depicts the proposed radial grouting plan. It is unclear to me how this effectively prevents seepage when used in combination with the seepage barrier. It would be preferable to align the radial grout lines closer together rather than 10' spacing and along the same alignment as the seepage barrier to more effectively minimize seepage. Consideration and discussion on the design intent here should occur.

1-0 Evaluation **Concurred**

The intent is to close any potential openings between the bottom of the seepage barrier and the grout curtain. One line of grout holes will extend into the seepage barrier and two additional lines of grout (one 10 upstream of the barrier and one 10' downstream). What spacing would the reviewer recommend?

1-1 Backcheck Recommendation **Open Comment**

In response to your question I would recommend a maximum 8-foot spacing between lines.

2-0 Evaluation **Concurred**

Recent meetings and discussions with the cadre responsible for performing risk assessment have resulted in elimination of the tunnel grouting.

2-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4595420	Geotechnical	Plans and Specs	n/a'	Plans, page CS701	n/a

The radial grout pattern has a large spacing between each grout line (spacings between grout lines of 5.5 to 14 feet at the start of the grout insertion point and 14 to 28 feet at the end of the grout line in section view). Consideration should be given to how effective the seepage control will be as it is my opinion the spacing pattern should be much closer to control seepage and will not provide an effective flow cut-off or increased flow path.

1-0 Evaluation **Concurred**

These are considered "primary holes". based on results of grout takes and pressure testing, additional holes may be required between these primary holes.

1-1 Backcheck Recommendation **Open Comment**

Thank you for the response. Consideration should be given to placing "Primary holes" at a closer spacing than 5.5 to 14 feet(14 to 28 feet and outer end of grout lines) since it is unlikely grout permeation will be much more than 2 to 3 feet radially. These spacings at the outer end of these lines are too large to grout effectively and control seepage and would require a substantial number of secondary holes to close off. It would be better to provide a more detailed plan as any gap will increase head and flow and is a issue that should be addressed in detail.

2-0 Evaluation **Concurred**

I do agree with the suggestion and would have adopted it into the plans and specs, however, recent meetings and discussions with the cadre responsible for performing risk assessment have resulted in elimination of the tunnel grouting.

2-1 Backcheck Recommendation **Close Comment**
Thanks for the response. I understand that this is being removed based on this risk assessment so the item may be closed without further comment.

IEPR Comment 4595422	Geotechnical	Plans and Specs	n/a'	Specifications/Section 03 37 29, Item 2.1.2.2, page 13	n/a

The table has several items that are cut off and unreadable and consideration should be given to modifying.

1-0 Evaluation **Concurred**
That was a formatting error and has been corrected.

1-1 Backcheck Recommendation **Close Comment**
I concur and this may be closed with no additional comment.

IEPR Comment 4595427	Geotechnical	Plans and Specs	n/a'	Specifications/Section 31 00 00, Item 3.12.3, page 13	n/a

Consideration should be given to modifying a moisture content range. I typically would prefer to set a range of moisture from a representative proctor test using a moisture range by plotting 0.95 x the maximum dry density and plotting this on the appropriate curve to determine minimum and maximum acceptable moisture content.

1-0 Evaluation **Non-concurred**
Understood, however, an acceptable moisture content range of -1 to +3% is fairly tight for the 95% density requirement (ASTM D698) specified considering the soils will be fine grained soils in this area. In addition, these areas of backfill (for utilities trenches in areas to be later paved) are rather minimal.

1-1 Backcheck Recommendation **Close Comment**
Thanks for the clarification. I concur with this assessment and this may be closed without further comment.

IEPR Comment 4595429	Geotechnical	Plans and Specs	n/a'	Specifications/Section 31 00 00, Item 3.15.5, page 13	n/a

In addition to one representative proctor test to determine optimum moisture and laboratory maximum density values at one test per 10,000 cubic yards it is advisable to have the contractor monitor gradations on a more frequent schedule to determine when a change in material occurs. It might be worthwhile specifying the Contractor shall monitor

gradation and plot these values on a gradation curve for comparison to the gradation of the proctor being used to verify the differences and whether a new proctor is needed.

1-0 Evaluation **Concurred**

Currently, paragraph 3.15.1 specifies that a minimum of one gradation test shall be performed per each 5000 CY stockpiled or in-place source material or as directed by the Contracting Officer. This will be revised to one test per each 4000 CY. Also, paragraph 3.15.1 shall be revised to specify that the Contractor shall monitor gradation and plot these values on a gradation curve for comparison to the gradation of the proctor being used to verify the differences and whether a new proctor is needed.

1-1 Backcheck Recommendation **Close Comment**

I concur and this may be closed with no additional comment.

IEPR Comment 4595431	Geotechnical	Plans and Specs	n/a'	Specifications/Section 31 56 00, Item 3.4, page 13	n/a

It is recommended that the Contractor also make additional observations for excavations to determine if tension crack formation or other signs of distress/deformation are noticeable as they are performing excavations.

1-0 Evaluation **Concurred**

Will revise text to include recommendation.

1-1 Backcheck Recommendation **Close Comment'**

I concur and this may be closed with no additional comment.

APPENDIX E

SEEPAGE BARRIER 100% P&S IEPR

Comment ID	Discipline	Section/Figure	Page Number	Line Number
IEPR Comment 4881653	Geotechnical	Plans	CG103	n/a

The legend defining work excluded from CWL uses a hatch pattern of lines which does not appear to match the plan view. Consideration should be given to modifying either the plan hatch or the legend hatch for consistency.

1-0 Evaluation **Concurred**
Will revise for consistency as recommended.

1-1 Backcheck Recommendation **Close Comment**
Closed without comment.

IEPR Comment 4881651	Geotechnical	Plans	CS115	n/a

Grouting, according to indicates casing through overburden and embankment bedrock is to be grouted to control seepage. The adjacent grout curtain seepage barrier has a top elevation of 977.0. The scale on the profile appears to be off as this elevation does not match the vertical scale. My primary question relates to the overburden. Is there a concern with seepage and piping through the overburden above the grout holes and whether this is an issue with the design?

1-0 Evaluation **Concurred**
Concur; the seepage barrier profile will be revised to the proper location and scale. The soil at the abutment is typically residual clays and silts and not susceptible to excessive seepage. In addition, flood events that will encounter these high-elevation soils have a very low frequency of occurrence.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4881649	Geotechnical	Plans	CS114	n/a

Sheet CS114 does not clearly indicate the primary, secondary and tertiary grout holes. While CS115 does indicate the intention is to have secondary and tertiary grout holes it is not clear on this sheet and I would recommend showing all primary, secondary, and tertiary grout holes on sheet CS114. It is also unclear if grouting starts at 0+00 or some distance off of 0+00G and if so what the distance is. Recommend better clarification as sheet CS115 also does not clearly indicate what this distance is or if it is the intention to have this offset from the grout curtain.

1-0 Evaluation **Concurred**
We will indicate primary, secondary, and tertiatry grout holes on CS114 as recommended. The spacing or offset between the end of the seepage barrier and the beginning (first grout hole) of the grouting work will be detailed on CS114 and CS115.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4881647	Geotechnical	Plans	CS114	n/a

Sheet CS114 does not clearly indicate the primary, secondary and tertiary grout holes. While CS115 does indicate the intention is to have secondary and tertiary grout holes it is not clear on this sheet and I would recommend showing all primary, secondary, and tertiary grout holes on sheet CS114. It is also unclear if grouting starts at 0+00 or some distance off of 0+00G.

1-0 Evaluation **Concurred**
CS114 will clearly indicate primary, secondary, and tertiary gout holes. The spacing between the seepage barrier and first (adjacent) grout holes will be detailed on this drawing.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4881646	Geotechnical	Plans	CS114	n/a

The legend for the work limits does not match the plan view. Additionally the grout line appears to extend outside the work limits. Please clarify whether the work limits should be adjusted based on the work required.

1-0 Evaluation **Concurred**
The symbology used in the plan view will be revised to match the legend as recommended. No adjustment to the work limits is required, grouting is permitted within the limited work limits referenced. Only limitations within these "limited work limits" are that the Contractor is required to provide and maintain access for other government contractor s and the Contractor is not allowed to remove any trees in this area.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4881644	Geotechnical	Plans	CS108	n/a

The section for station 63+25B Left Abutment Impervious Blanket requires excavation and key-in with what appears to be a vertical cut in excess of OSHA requirements. How will this be cut and meet safety standards?

1-0 Evaluation **Concurred**

The requirements for the impervious blanket has been revised substantially. We have added additional details and clarification regarding the limits and required key-in of impervious blanket to the impervious core of the dam and seepage barrier. In summary, we now require placement of seepage barrier instead of compacted impervious fill, perpendicular to the primary seepage barrier. The length of the secondary seepage barrier is approximatley 15 as measured from the centerline of the primary barrier. We ancticpate that a majority of the material adjacent to the secondary seepage barrier will need to be removed for construction of the primary seepage barrier. Therefore, construction of the secondary seepage barrier should be possible using the same methods as is used to construct the primary. The impervious blanket of material will be keyed into the primary seepage barrier, secondary seepage barrier, and impervious core as detailed on the revised sheet. Contractor is required to expose the impervious core of the dam to accomplish the "keying" in of the impervious blanket material to the impervious core of the dam. It is anticipated that this will be accomplished with shallow excavation extending up the slope of the dam to elevation 982.0. The contractor has the flexibility to determine the actual alignment that will be used to make connection (or key-in) between the impervious blanket and the impervious core of the dam. An updated drawing can be made available at your request.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4881643	Geotechnical	Plans	CS115	n/a

On both the Waterline Trench Detail and the Waterline Under Paved Area Detail the dimension lines appear to be missing for the required minimum material to be placed under the pipe.

1-0 Evaluation **Concurred**

Dimensions detailing required depths and thickness of bedding material will be shown in both details.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876879	Geotechnical	Section 31 57 00: Verification Drilling and Testing; Sub-section 3.2.7, Preservation of Core Test Samples	9	n/a

This section starts as: "The core shall be washed, wrapped in aluminum foil, thin plastic wrap, or cheese cloth and then sealed by applying paraffin wax, microcrystalline wax,

50/50 mixture of paraffin and microcrystalline wax prior to placing the core in the core box". The bottom part of the section states: "The minimum length of core that is preserved from each boring shall be no less than 2.5 times the core diameter". It is not clear from this section if the wrapped core is the core to be used for laboratory testing. If so, a core sample approximately 2.5 times the core diameter appears to be insufficient for testing purposes and may not represent the variability for the entire core from a given boring.

1-0 Evaluation **Concurred**

The text will be revised to clarify that these preservation requirements are only for core samples for laboratory testing. See paragraph 3.2.6 (Selection of Test Samples) for discussion of core sample selection and core sample selection with respec to variability.

1-1 Backcheck Recommendation **Close Comment**

The purpose of my comment was to ensure that appropriate length of the core required for laboratory testing will be preserved, not the length equal to 2.5 times the diameter of the core (as this may not be sufficient). I am closing the comment with an understanding that a core length equal to "2.5 times the diameter of the core" will not be a restriction on the core length preserved for laboratory testing.

IEPR Comment 4876877	Geotechnical	Section 31 63 29: Parapet Wall Piles; Sub-section 3.6 – Installation of Pile.	8	n/a

The section states: "Upon acceptance by the Contracting Officer of the excavation, the contractor shall remove any water in the hole and maintain in a dry condition during placement of concrete in the hole by conventional means. If the Contractor intends to use tremie concrete placement methods the water is not required to be removed from the hole. The Contractor shall install the pile in the casing in accordance with Section 05 12 00 STRUCTURAL STEEL. The Contractor shall support the piles ------". This part of the section implies that concrete will be placed before pile installation whereas the next Sub-section (3.7) on the same page states that the Contractor shall pump concrete into the hole after the pile is accurately installed in the drilled shaft. This needs some clarification.

1-0 Evaluation **For Information Only**

Sub Section 3.6 and 3.7 have been edited as follows: 3.6 INSTALLATION OF PILES Upon acceptance by the Contracting Officer of the excavation, the Contractor shall install the pile in the casing in accordance with Section 05 12 00 STRUCTURAL STEEL. The Contractor shall support the piles with the temporary pile support which shall accurately position the pile in its intended final location. The pile shall be installed with centralizers or other means to be determined by the Contractor to prevent the pile from moving laterally during the concrete placement in the drilled shaft. The Contractor shall thoroughly clean the pile to assure good bond with the concrete. The Contractor shall set the steel pile into the open hole and use a template to hold the pile in alignment. 3.7 INSTALLATION OF CONCRETE The Contractor shall pour concrete into the drilled shaft after the pile is accurately installed. Concrete shall be placed by the pumping method. Contractor shall maintain a dry condition in the drilled shaft during the concrete placement. The Contractor shall use approved means that will prevent segregation of the material during the pumping process and shall be performed in a continuous method until the entire area is filled. Operate the pump so a continuous stream of concrete without air pockets is produced. Placement operations shall be as necessary to produce sound, durable concrete foundation shafts free of defects. Control the initial rate of concrete placement so not to lift or displace the steel pile. Flowing water that prevents proper placement of concrete shall be controlled before concrete placement begins. Do not begin placing concrete until the pump line orifice is at the shaft base elevation. The preferred concrete placement procedure is to maintain the outlet end of the pumping system at approximately 10 feet below the top of the fresh concrete. Concrete shall be continuously placed by methods that ensure against segregation and dislodging of excavation sidewalls, and shall completely fill the hole. Removal of the temporary casing shall begin within one hour from beginning of concrete placement in the cased portion of the shaft. Extract casing at a slow, uniform rate with the pull in line with the shaft axis.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876875	Geotechnical	Section 31 56 00; Sub-section 3.3, Excavated Material.	13	n/a

The sub-section states: "Boulders excavated from the trench shall not be used in the Seepage Barrier backfill material unless their non-detrimental use is fully demonstrated in the Seepage Barrier Construction Plan and approved by the Government". A particle size should be specified to define the boulders. Also, in the opinion of this reviewer, it will be better not to use the boulders at all as their presence will tend to decrease the homogeneity of the barrier.

1-0 Evaluation **Concurred**

Definition of a boulder will be included in subsection 3.3.

1-1 Backcheck Recommendation **Close Comment**

If the boulders are used in barrier construction, this reviewer suggests that their largest dimension should not exceed half (preferably 1/3) the thickness of the blanket.

IEPR Comment 4876874	Geotechnical	Section 31 56 00: Seepage Barrier Construction; Sub-sections 1.9.1 (Borings) and 1.9.2 (Quality).	11	n/a

Sub-section 1.9.1 states: "If on-site soils are utilized for Contractor's Seepage Barrier Backfill Mix Design (reference paragraph BACKFILL MIXING AND PLACEMENT below), borings shall be drilled at four separate locations (on 100-foot centers) within the Seepage Barrier DEMONSTRATION SECTION limits". Sub-section 1.9.2 states: "If the Demonstration section does not pass the performance criteria, the Contracting Officer may direct the Contractor to make revisions to the construction method and/or Seepage Barrier backfill mix design and install additional Demonstration Sections at no additional cost to the Government until the Demonstration Section passes the acceptance criteria". In the opinion of this reviewer, the Contractor should be asked also to drill borings prior to construction of additional Demonstration Sections. This will provide additional information about the variability of on-site soils, if used for mix design.

1-0 Evaluation **Concurred**
Revised as requested.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876872&	Geotechnical&	Section 31 32 23; Sub-section 3.9.1.1 –Drilling logs.	29	n/a

The section states: "The contractor shall record pertinent information during the drilling of each grout hole. Information shall include but not limited to: hole number, station, top of hole, -------". This section includes everything but geology. It is important to specify that drill logs will include geologic information. Elsewhere in Section 31, it is stated that a professional geologist will log the holes. The geologist should include geologic information in the logs.

1-0 Evaluation **Concurred**
Concur; text will be revised to include geologic information. However, only limited geologic information can be obtained since grout holes will be drilled without obtaining core samples.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876870	Geotechnical	Section 31 32 23; Sub-section 3.6.5 – Additional Exploration.	22	n/a

This section starts with: "As the drilling and grouting work progresses, it may develop that conditions are such that all or parts of the foundation already grouted require additional exploration". However, the section does not specify the conditions that may lead to such a decision nor does it state the indicators of such conditions. This requires some clarification, including who will be the in charge of evaluating the conditions and making the decision about additional exploration and grouting.

1-0 Evaluation **Concurred**
Concur, text will be added such that additional exploration shall be directed by the Contracting Officer s Representative. However, numerous conditions may be encountered that would warrant additional exploration, which would not be practical to specify and would be difficult to quantify. In addition, specifying conditions could also limit other overlooked conditions. The decision will be experienced based, taking into consideration the conditions encountered in the field.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876868	Geotechnical	Section 31 32 23; Sub-section 3.6.3 – Upstage Grouting.	22	n/a

The sub-section states: "The upstage grouting method will be the preferred method of drilling and grouting where practical". The rationale for upstage grouting being the preferred method of grouting is not clear to me. Downstage grouting allows upper strata to be consolidated so that higher grouting pressures could be used at deeper levels without uplift problems.

1-0 Evaluation **For Information Only**
Upstage grouting is typically cheaper since drilling for a hole can be completed from a single drill setup while minimizing the amount of redrilling through previously placed grout. Higher grouting pressures will still be obtained at the deeper grout zones by placing a packer at the base of the upper grout zones.

1-1 Backcheck Recommendation **Close Comment**
I understand the cost aspect. However, downstage grouting may still be a better option for a highly fractured, weak rock zone extending to the ground surface.

IEPR Comment 4876866	Geotechnical	Section 31 32 23; Sub-section 3.5.3– Grout Hole Rock Drilling.	20	n/a

The first paragraph of this sub-section states: "Where conditions are encountered at the scheduled bottom of hole depth that would indicate grout takes are likely, then the drilling shall continue until the boring penetrates 10 feet of rock where minimal grout takes are anticipated, or until cessation of drilling is directed by Contracting Officer's Representative". The basis for judgment about "grout takes are likely" or "where minimal grout takes are anticipated" is not clear. What tests or indicators will provide the basis for such judgments?

1-0 Evaluation **For Information Only**
These judgments will take into consideration the various conditions encountered in the field. One condition is if sufficient drill water loss or voids are encountered during the drilling process. There are too many of these variables to be practically listed in the specifications. The judgment will be experienced based, which is one reason why experience is a specified requirement. The hole will be deepened to include the entire bedrock feature.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876863	Geotechnical	Section 31 32 23; Sub-section 3.5, Drilling Procedures; Sub-section 3.5.1 – Grout Hole Depths, Inclinations, and Tolerances.	19-20	n/a

The section states: "The Contractor shall exercise care in the alignment of the drilling equipment to ensure that the initial alignment of the hole is within one degree of the theoretical alignment in both direction and inclination. In the event that the required tolerance cannot be achieved, the Contractor shall provide appropriate drilling equipment or modify existing equipment and procedures to produce alignments within the specified tolerances". It is not clear who will monitor hole-alignment. This task needs to be overseen by the Government or its Representative.

1-0 Evaluation **Concurred**
Concur, additional text will be added specifying the Contractor s responsibility to measure alignment with accessibility to the Government for quality assurance testing. Typically, the contractor will unitize a level to determine initial inclination.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876859	Geotechnical	Section 31 32 23; Sub-section 2.5 –& 13& Sand, Part		n/a

The sub-section states: "The percentage of surface moisture, in terms of saturated surface-dried sand, shall be determined in accordance with ASTM C70 or other method giving comparable results". The purpose of measuring surface moisture is not clear nor is it clear if there will be a specification in this regard.

1-0 Evaluation **For Information Only**
The purpose of this requirement is to assist in mixing grout that is consistent, to help determine if adjustments are needed to the water quantity. Sand moisture criteria will not be specified.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876856	Geotechnical	Section 31 32 23, Drilling and Grouting; Sub-section 1.3 – Submittals.	7	n/a

Under Grouting Drawings; EC, it is stated: "Separate sheets shall be provided for 1) section view(s) of drilling including lost tooling, and summary geologic information 2) -----". It should be specified as to what type of geologic information to include in the summary sheet.

1-0 Evaluation **Concurred**
Concur, text will reference paragraph "CADD Profiles and Sections."

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876854&	Geotechnical&	Section 31 11 00, Clearing and Grubbing; Sub-section 3.6 – Disposal of materials.	4	n/a

This sub-section specifies how to dispose of non-saleable materials such as tree stumps that cannot be burned. However, it does not specify how the ash residue from burned materials will be disposed of, if the Contractor chooses to burn some of the waste material.

1-0 Evaluation **Concurred**
The spec section has been revised to require the ash residue be disposed of in the onsite borrow area and covered with a minimum of two feet of random fill.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876850	Geotechnical	Section 31 00 00; Sub-section 3.9.2& – Compaction.	13&	n/a

The sub-section states: "Except for paved areas, compact each layer to at least 90% of laboratory maximum dry density according to ASTM D698". In my opinion, water content should be also specified. It is specified elsewhere.

1-0 Evaluation **Concurred**
Revised as requested.

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876847	Geotechnical	Section 31 00 00; Sub-section 1.8 – Utilization of Excavated Materials.	8	n/a

This sub-section states: "Satisfactory (fine-grained and coarse-grained) materials removed from excavations shall be used, insofar as practicable, in the construction of left abutment impervious blanket, fills, embankments, sub-grades, bedding (as backfill), and for similar purposes". This statement may imply that both coarse-grained and fine-grained materials can be used to construct impervious blanket.

1-0 Evaluation **Concurred**
The sentence will be revised as follows: "Satisfactory (fine-grained and coarse-grained) materials removed from excavations shall be used, insofar as practicable and where appropriate, in the construction of left abutment impervious blanket, fills, embankments, sub-grades, bedding (as backfill), and for similar purposes."

1-1 Backcheck Recommendation **Close Comment**

IEPR Comment 4876841&	Geotechnical&	Section 31 00 00: Earthwork; Sub-section 1.5 – Definitions.	6-7	n/a

On p. 6, the document defines satisfactory coarse-grained materials as GC, GP-GC, GM-GC, SW, SP, SM, SW-SM, SC, SW-SC, SP-SM, and SP-SC (1.5.1) while satisfactory fine-grained materials are listed as CL, CL-CM, and ML (1.5.2). On page 7, the document states that unsatisfactory materials are those which do not comply with the requirements for satisfactory coarse-grained or fine-grained materials (1.5.3). The paragraphs under 1.5.1 and 1.5.2 also state that requirements for gradation testing frequencies are provided in paragraph FILL AND BACKFILL MATERIAL GRADATIONS. Grain size distribution (gradation) can be an important requirement for coarse-grained soils, but not for fine-grained soils. Atterberg limits may be more relevant for fine-grained soils. Also,

some of the coarse-grained soils listed above as satisfactory materials may not meet the gradation requirements.

1-0 Evaluation **Concurred**

Paragraph 3.16.1 will be revised to include Atterberg limit testing requirements for plastic, fine-grained soils.

1-1 Backcheck Recommendation **Close Comment**